Risk Management in Deep Excavation

Mehrdad Rashidi Tabar

Submitted to the
Institute of Graduate Studies and Research
in the partial fulfillment of the requirements for the degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
June 2016
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. MustafaTümer
Acting Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Assoc. Prof. Dr. Serhan Şensoy
Acting Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion, it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Dr. Tolga Çelik
Supervisor

Examining Committee

1. Prof. Dr. Tahir Çelik
2. Assoc. Prof. Dr. Khaled Marar
3. Assoc. Prof. Dr. İbrahim Yitmen
4. Dr. Tolga Çelik

ABSTRACT

Urban decision-makers such as private owners or public section (e.g. municipality) are encouraged to optimize the use of land in underground which has opportunity of economic or social benefits such as Metro stations, multi-story commercial buildings with parking lots or shops in underground. These decisions can be endangered in construction processes. There are few scientific and formal reports about irrecoverable damages, fatalities and geotechnical engineering researches to predict side soil displacements, strength, and their allowable quantities to reach an improved design technique for retaining supports and soil stability. But construction processes may face possible risks such as geotechnical, structural and productivity risks which have additional cost and increased duration. Therefore, risk management in deep excavation is a crucial field to study and there isn’t any collective scientific resource on that subject. This thesis have prepared a collective resource from related scientific fields, summary of case studies, and case histories in deep excavation to be use in risk management. The objectives are geotechnical, structural, and construction productive risk identification, risk occurrence probability, and risk consequence that are the parameters of risk assessment, which is require for risk response plan in risk management. In this manner, construction methods and equipment in deep excavation are summarized and classified by a proposed facade vision. A deterministic method is proposed for geotechnical risk occurrence probability estimation based on factor of safety concept. A method is proposed for classifying deep excavation damages and estimating expected internal, external, and accidental damages as a consequence in deep excavation is proposed. In order to identifying risks, site investigation and underground identification is overviewed and a model based on range of parameters for classification and identification of clays, granular soils, and intermediate soils is developed which are required for sensitivity analysis as a method of risk assessment. For risk identification and analysis, different geotechnical failure modes, structural failures and their effects on adjacent land and building such as settlement, cracks, is overviewed and collected. The repair state classification and dewatering effects are overviewed, and collected as well. Method of estimating expected internal, external, and accidental induced-damages in deep excavation is proposed and compared in each stage. Also risks and uncertainties in productivity such as production rate, work duration, and unit cost is discussed for considering the preparation of response plan in construction of deep excavation. Geotechnical risk occurrence probability estimation and risk consequences in deep excavation are innovative proposed method. Site geotechnical investigation and identification for risk management in deep excavation is innovative expanded method. Cost risk management in deep excavation is innovative expanded method.

Keywords: Deep excavation, risk occurrence probability, risk consequence, geotechnical risk, Cost and duration risk, production rate risk, risk response plan

ÖZ

Őzel sectőr veya kamu yőnetimleri (őr: belediyeler) gibi kentsel karar vericiler, Metro istasyonlari, yeraltinda dűkkan ve otopark alanli ticari binalar yapimi, economic ve sosyal fayda firsatlari vermesi bakimindan yeralti alanlarinin optimizasyonu teşfik edilir. Bu kararlar inşaat işlemlerinde tehlikeye yol açabilir. Istinat desteklenmesi ve sabitlemesi için gelişmiş tasarim tekniklerine ulaşmak için, toprak deplasmani, mukavemeti ve onlarin kabuledilebilir miktarlarini tahmin etmekte kullanilaçak, telafisi olmayan hasarlar, őlűmler ve geoteknik műhendisliği çalişmasi hakkinda az sayinda bilimsel ve resmi rapor var. Fakat inşaat işlemleri, ek maliyet ve uzatilmiş inşaat sűresi gibi sorunçlar doğurabilecek, geoteknik sel, yapisal ve verimlilik riskleri gibi bazi olasi risklerle karşilaşir. Bu yűzden, derin kazilardaki risk yőnetimi çok őnemli bir çalişma olanidir ve bu konuda toplu bir bilimsel kaynak yoktur. Bu tezle, risk yonetiminde kullanilmak űzere derin kazilar ile ilgili bilimsel alanlardan, őrnek çalişmalardan ve őrnek geçmiş olay kayitlarindan faydalanilarak bir toplu kaynak hazirlanmiştir. Amaç, risk yőnetimi için, geotekniksel, yapisal ve verimlilik risk tanimlamasi, ve risk degerlendirmesini oluşturan, risk oluşma olasiliği ve sonucu parameterlerini hazirlamaktir. Bőylelikle, derinkazilardaki anşaat methodlari ve ekipmanlari bir őnerilmiş cephe vizyonuyla őzetlenmiş ve siniflandirilmiş oldu. Emniyet faktorű koseptine dayanarak, geotekniksel risk oluşma olasiliği için bir deterministic method őnerildi. Derin kazi sonucu olarak, derin kazi hasarlari siniflandirmasi ve beklenen içsel, dişsal ve kaza neticesi olan hasarlar için bir method őnerildi. Bir risk değermelendis metodu olarak duyarlilik analizi için gerekli olan risk tanimlamasi için, saha incelemesi ve yeralti tanimlamasi gőzden geçirildi ve kil, granul toprak ve orta topraklar

siniflandirmasi ve tanimlamasi için parameter araliklarina bağli olarak bir model geliştirildi. Risk tanimlamasi ve analizi için farkli geoteknik başarisizlik modelari, yapisal başarisizlik ve onlarin komşu arazi ve binalar űzerindeki oturma, çatlama gibi etkileri gőzdan geçirildi ve toplandi. Onarim durumu siniflandirmasi ve su tahliye etkileride gőzdan geçirildi ve toplandi. Derin kazilardaki, beklenen içsel, dişsal ve kaza sebepli hasar tahmini metodu őnerildi ve her seviyede karşilaştirildi. Derin kazi inşaasi müdahale plani hazirlanmasi gőz őnünde tutularak, üretim hizi, iş sűresi ve birim maliyet gibi verimlilik konusundaki riskler ve belirsizlikler de tartişilmiştir. Derin kazı jeoteknik risk oluşumu olasılığı tahmini ve risk sonuçları yenilikçi önerilen yöntemdir. Derin kazı risk yönetimi için site jeoteknik araştırma ve kimlik yenilikçi genişletilmiş bir yöntemdir. Derin kazı Maliyet risk yönetimi, yenilikçi genişletilmiş bir yöntemdir.

Anahtar Kelimeler: Derin kazı, risk olay olasılık, risk sonucu, jeoteknik riski, Maliyet ve süresi riski, üretim oranı riski, risk müdahale planı

To My Family

ACKNOWLEDGMENT

I would like to express my deep appreciation to my supervisor Dr. Tolga Çelik for his inspiration and guidance throughout this work. This thesis would not have been possible without his support. I would also like to thank the examining committee members, for taking time to review my thesis.

I want to express my sincerest gratitude to Assoc. Prof. Dr. Huriye Bilsel who trained and helped me with various issues in field of geotechnical engineering during the thesis.

Assoc. Prof. Dr. Serhan Şensoy Acting Chairman of the Department of Civil Engineering, Eastern Mediterranean University, helped me with various issues during the thesis and I am grateful to him. I am also obliged to Prof. Dr. Tahir Çelik for his coordinate to Spring Mall construction project in Northern Cyprus to see and research during my thesis as well as training in field of construction management. Besides, a number of friends had always been around to support me morally. I would like to thank them as well. I would also like to thanks to other staff members of the Department of Civil Engineering for their help.

I would also like to thanks to Mr. Hakan Yildiz (Senior Civil Engineer) for his assistance to gather data for a case study in North Cyprus, and Mr. Mustafa Kokel (Senior Civil Engineer) for his technical translation of őz in Turkish language.

Finally, I would like to give my heartfelt thanks to my lovely wife Elham for her support and understanding during my study.

PREFACE

This thesis includes the collective subjects required for risk management in deep excavation as well as proposed formula for estimating probability of geotechnical risk occurrence, and proposed method for expected damages as a risk consequence. A case of construction cost risk and a case of geotechnical risk assessment and risk response plan is studied for deep excavation. Discussions on case histories and construction productivity of deep excavation for risk identifying, assessment, and management are prepared as considerations. It seems this thesis can be a collective framework for future researches about risk management in deep excavation.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1

INTRODUCTION

1.1 Background of research

Urban decision-makers such as private owners or public section clients (e.g. municipality) are encouraged to optimum use of land in underground which has opportunity of economic or social profits such as Metro stations, multi-story buildings with parking lots and shops in underground, open-cut subway tunneling in soft underground, some defensive sites (e.g. underground shelters), riverside, or costal beach. These decisions has endangered in construction processes so that there are few scientific and formal reports about irreparable damages and/or fatalities even 21 casualties in a failure case [1]. In order to deal with problem, geotechnical engineering researches to predict side soil displacements, strength, and their allowable quantities to reach an improved reliable design technique for retaining supports [1, 2, 3, 4, 6, 7, 8, 9].

Although geotechnical engineering researches to predict side soil's and excavated floor's displacements, strength, and their allowable quantities but construction processes such as excavation stages or trenching for retained wall may face possible risks such as geotechnical, and productivity risks which have additional cost and increased duration of implementation [1]. Due to mentioned, risk management is crucial field for study and apply on deep excavation construction which there isn't any collective scientific resource about that. Risk management needs a methodology to study situation and risk identification and assessment which can lead to appropriate response plan to how mitigate or eliminate the problem.

In this study a method is proposed to risk identifying, estimate geotechnical risk occurrence probability, and risk consequence. An appropriate risk management methodology is decreasing the risk probability and impacts on project objectives with steps such as: firstly grasp increase by gathering existence science, well-documented recent case histories, lessons learned, identifying potential risks, and risk register by output of risk identification, then analyzing the situation of each project based on data existence, and finally preparing risk response plan and monitoring and controlling risks which lead to mitigate, eliminate, deal with, or avoid the problem.

After applying the methodology there are possibilities to estimate geotechnical risk occurrence probability, and expected risk consequence if data is existed which depends on situation and investigation planning and accuracy, otherwise uncertainty is existing which determine for intensive mitigating, dealing and controlling. In addition the methodology can check the design for construction processes and improve it, otherwise can redesign excavation supporting system and stages. Even by applying the methodology it is possible to design deep excavation according construction processes.

1.2 Scope and objectives

Generally, excavation is a construction activity after designing (and demolishing existent building) and before loading, hauling, and dumping of soils, rocks, or demolished materials. The need to increasing the depth of excavation due to one to five and more basement below surface leads to risk of stability of sidewall and base soil issues, safety shoring risks, accidents risk, and productivity risks especially in urban. If there is enough space for planning and excavation without damaging other structures or adjacent limitations, the side of excavation may be sloped or benched to prevent ground failure or adjacent land deformation, and cracking but rarely comes in urban. Excavation sidewalls may be collapsed or deformed and due to that it affects adjacent structures, facilities, estates, instruments, equipments, and/or humans' life. Occurrence of risks in projects within deep excavation results occasionally in significant losses of lives and properties, additional costs, and delay in project completion. The sort of knowledge, resources and activities is required for risk management and improving conditions to reduce the probability of risk occurrence risk consequence which can be damages due ground failures or adjacent building or properties settlement, additional cost of unfavorable production, cost due to additional material consume, cost due to material price increase, and other project objectives. Since construction projects are unique, the issue of risk management in deep excavation for each project has to be studied separately. However this study tries to show it is possible to prepare risk comprehensive framework for range of projects include deep excavation in urban.

The objectives for this thesis are listed below:

- To perform site geotechnical investigation and identification
- To carry out geotechnical and constructional productive risk identification and register
- To conduct risk occurrence probability estimation
- To estimate consequences of risk
- To undertake risk response plan

1.3 Methodology in brief

The methodology of this thesis in brief is based on underground data which obtains by field or laboratory investigation tests, underground identification based on proposed classification method, identification of failure modes or failure effects as risk identification which is overviewed, estimation of each identified risk occurrence probability based on proposed method, estimation of risk consequence such as expected internal or external or accidental damages based on proposed method gives rise to risk response plan. It is possible that the existence design is insufficient which leads to redesign by applying risk assessment. Real geometry and underground soils from geotechnical engineering researches case study which predict the wall deflection and ground settlement is used in risk occurrence probability estimation and expected risk consequence as an example for clarity (see 9.6). A case study from Cyprus for representing risk of planning on cost and scheduling and risk of adjacent buildings is prepared (see 9.7). The Bell's formula in geotechnical engineering is developed to evaluate cantilever retaining wall length require to mitigate sliding which leads to quadratic equation and cost risk due type of underground soil (see 9.2, 9.3, 9.4). Potential damages due to sliding in case of diaphragm wall constructing are described (see 9.5).

1.4 Achievements

After applying the methodology there are achievements to estimate geotechnical risk occurrence probability which is need to estimate risk consequence such as expected damages, or reach to scientific approach to eliminate risk or deal with risk. Also by applying the methodology it is possible to achieve to reach an appropriate grasp for how managing the uncertainty conditions to save cost and time of construction processes which can reflect in contracts.

This thesis is a collection to answer the questions such as what is the difference between deep excavation and ordinary excavation, which methods and equipments are more effective for a specific deep excavation, what is that cost in comparing ordinary excavation, what effects are there by deep excavation, what factors affect deep excavation, how can identify and register risk, how can assess risk, what are cost risks, how can prepare risk response plan and strategy, how deep excavation can remedy special failures such as liquefaction, and how is the comparison of internal and external damages trends in terms of excavation stages.

1.5 Guide to thesis (chapters)

This study is divided into ten chapters. Each chapter has introduction in order to asking questions which is presented initially except chapter ten. Also a brief conclusion at the end of each chapter is prepared except chapters nine and ten. Target is risk management in deep excavation which depends on deep excavation knowledge area and risk management expertise. In other word it is request to present essence of deep excavation earlier than risk management and after that risk identification, assessment and response as a steps of risk management jointly with underground related knowledge.

This study initially focuses on overview to deep excavation in chapter 2. It includes a simple and new definition of deep excavation, excavating without support, and supporting system for deep excavation with alternate methods based on steps facades. In this vision, supporting system is divided to three categories. Retaining walls as a first facade supporting system in kind of reinforced concrete diaphragm wall, bored pile walls as contiguous, secant, and tangent, steel sheet pile, and soldier beam and lagging with their construction methods and equipments are briefly described in the first category. The second category is secondary supporting system which added in one level struts or anchors on wall with two stages for excavation that the construction methods and equipments are explained in short. The third category is multi-propped multi stages supports with multi stages excavation which the methods, and equipments are described. General geometry of deep excavated sites as case history is gathered and discussed and monitoring instruments are mentioned too. Chapter 2 tries to prepare and present a collection of existence methods, technologies, and techniques for deep excavation managers, engineers, contractors, clients, and owners.

Risk definition and category is overviewed and expanded in chapter 3. It includes risk definition, categories, expected damage at risk which has a proposed method, uncertainty, and risk and decision-making. Expected damage at risk is divided into internal, external and accidental and two objective formulas are proposed for additional cost and scheduling increasing.

Methodology for risk management in deep excavation is presented in chapter four. The methodology has steps such as grasp increase by studying of existence science and well-documented case histories and lesson learned, identifying potential risks, risk register, analyzing situation and risk assessment, create risk response plan, and control risks through the project by monitoring. Analyzing situation and risk assessment includes: the common probable causes of risk, geotechnical risk occurrence probability, and risk consequence. Furthermore this thesis has more emphasis on Analyzing situation and risk assessment in deep excavation. A formula is proposed for estimating geotechnical risk occurrence probability based on factor of safety (FOS). A vision and method is proposed for risk consequence based on geotechnical risk occurrence probability and damages which divided into three cases. Although new version of the important building codes (e.g. EUROCODE) recommend use of limit state method with partial coefficient due to probability influence in design of geotechnical or structural mechanisms instead of safety factor which had been in early versions (e.g. BS 8002) a recommended engineering sure limit (not absolutely) but it seems easy to use of safety factor as a random variable especially for risk management in deep excavation which can include not only safety of construction management but also in design or checking the design is considerable. Of course, it is possible to calculate factor of safety from limit state method with partial coefficient as a sure limit with one or two additional multiplication or division and there isn't any intention to flaws limit state method with partial coefficient for design.

Subsurface conditions are requiring to investigation, identification, and classification. Ground identification and classification, and site investigation are not only overviewed in chapter five but also an approach about underground soils identification based on low and high limits for soil parameters is proposed which is require in sensitivity analysis in risk assessment. The importance of site investigation and identification in deep excavation calls for specific focusing on it. The range of underground soil data for clays, granular, and intermediate soils with a simple approach are gathered which can be used in estimating of outcome of uncertain happening in deterministic method. The direct use of the ranges without any tests to investigation for many reasons is not logical because leads to design and construction in the worst case. The worst case due to uncertain condition could cause unnecessary plan and even the severe unpleasant consequences, which may be irreparable. In that reason, site investigation process and details is overviewed in chapter five which includes arrangement of points and depths, sampling, water table, characteristic property, and site investigation cost.

Ground and supports failures as source of risk, which identified risks in deep excavation, are overviewed in chapter six. It includes analyzing of sliding, overturning, bearing capacity, basal heave, upheaval, liquefaction, heaven, piping, sand boiling, and another ground failure modes which collected altogether. This chapter is important for expected internal damage estimating. Also probability of geotechnical risk occurrence probability can be estimated by basis of chapter six.

Ground movement, settlement, their limits and building damage as effect of deep excavation on adjacent land, building and properties is overviewed in chapter seven. This chapter is important for estimating external risk occurrence and expected external damage as risk consequence. .

High groundwater table in site leads to dewatering for deep excavation which its effects have potential risk is overviewed in chapter eight. Chapter eight in locations with water table level upper than final level of excavation is important because of dewatering negative effects in deep excavation which caused settlement of adjacent buildings' foundations and damage.

Illustrative examples, considerations on case histories, case study, and considerations on productivity and work duration are presented in chapter nine. This chapter has studied firstly comparison of two different underground situations on site chosen, and design which effect is significance. Potential risks in constructing diaphragm wall as risk identification is proposed next. For a top-down multi level strutted method identified geotechnical failures as a risk is analyzed and probability of risk occurrence for each situation is calculated and possible and expected internal, external, and accidental damages is estimated which are necessary to prepare appropriate responses plan and a geotechnical risk response plan is proposed. A case study of multi-level anchored contiguous bored piles wall which focuses on some cost risk due to construction plan in Northern Cyprus is presented which shows samples of cost and budget risks in deep excavation. Some considerations on case histories or others case studies is proposed so that the sharing of risky factors effects on consequence of outcomes is surveyed. Also risk of production rate and duration based on the conceptual framework for the preparation of risk response plan in deep excavation is investigated for grabbing and excavating and the effect of risk on unit cost is estimated.

Chapter ten is conclusion that includes fifty five paragraphs which of each are summary of early chapter's main results.

Chapter 2

DEEP EXCAVATION

2.1 Introduction

There are questions such as what is deep excavation, when and where the excavation is deep excavation. Also what is supporting system and its alternatives, which alternatives could be suitable for a certain situation generally, what methods and equipment are known and proper to its construction, and what relationship between site geometry and appropriate supporting technique is? This chapter tries to describe appropriate answers to the above questions.

The excavation is function of subsurface conditions (soil and water or liquid), and digging equipment, but successful deep excavation is function of supporting system, and environmental impacts in addition. Supporting system includes retaining walls (reinforced concrete or steel), secondary supports in approximately one level such as struts, tie back, slab of main structure over the piles with staging excavation, and multi-propped multi-level supporting system. Deep excavating construction may require backhoe (for guide lower depth trench to clamshell application), appropriate clamshell for deep trench digging put into diaphragm walls, and/or drilling rig for bored pile walls, and appropriate hydraulic excavator relating to volume of each excavating stage into site with appropriate trucks for hauling, plus supporting system to sidewall or adjacent properties safety depend on plan and situation.

Selection of type of supporting system and construction method plan depends on site geometry, available way to site, available technology, local tradition and rules, available labor force, type of underground soil, depth of excavation, and generally speaking situation. Therefore there are necessities to increase grasp and study about the mentioned subjects to achieve a deep excavation safe implementation.

This chapter contents:

1- Deep excavation definition

2- Vertical cutting without any supports

3- Supporting system for deep excavation

3.1- First facade supporting system for deep excavation (Cantilever retaining walls)

3.1.1- Diaphragm wall construction with clamshell

3.1.2- Bored pile wall Construction with drilling auger machine

3.1.3- Excavator (hydraulic backhoe)

4- Second facade supporting system for deep excavation (one level struts, and/or anchors)

4.1- Anchor, tie back, and soil nailing

4.2- Strutting

5-Multi-level secondary supports with multi stage excavation (struts, anchors, top-bottom method, and combination of mentioned methods)

6- General geometry of deep excavation sites

7- Monitoring instruments and equipments

8- Index for deep excavation definition

2.2Deep excavation definition

Excavation depths more than 3 to 6 meters, necessitate particular forecast for hold up normally [10]. As an ordinary definition, deep excavation could be known an excavation with depth in more than 1.50 meters on soft clay, 3.0 meters on medium clay and generally more than inherent safe height of different types of soils (see table 5.2) under surface so that the vertical side of excavation or excavation floor probably tends to instability. This definition is based on failure of vertical cutting side. There may be another approach based on effect of excavation on adjacent properties such as cracks, settlement or deformation in land or foundation of buildings. Also based on human occupancy health and safety, excavation more than height of sole to throat of a normal human could be deep excavation due to its risk of failure of vertical cutting side and fall down on human.

2.3 Vertical cutting without any supports

While a cut is made, the soil at the vertical surface inclines to dilate and move into the cut zone. The variation of moisture content of soil or internal water table could affect an excavation vertical surface. Depending on kind of soil and absent surcharge, stable or safe vertical height of soil after excavation may be observed in short term relatively. For example, the approximate average safe vertical height of soft clay, medium clay, stiff clay, and very stiff clay are 1.5, 4.5, 9.0, and 18 meters in short term after excavation, respectively (see table 5.2). Failures such as sliding, subsidence, toppling, heaving, boiling (due to water table), and also tension cracks owing to deformation of soil in cutting are different modes that could be probably occurred due to vertical excavation without any supports. Figure 2.1 shows different modes of failure or effect of failure in vertical cutting without any supports [10]. The primary influence zone may be two times of excavation depth (Hsieh and Ou 1998) and more. Beside of mentioned failures, there are other failure modes such as collapse due to nearby excavation, environmental getting wet and drying, and failure due to liquefaction in vertical excavation without any supports.

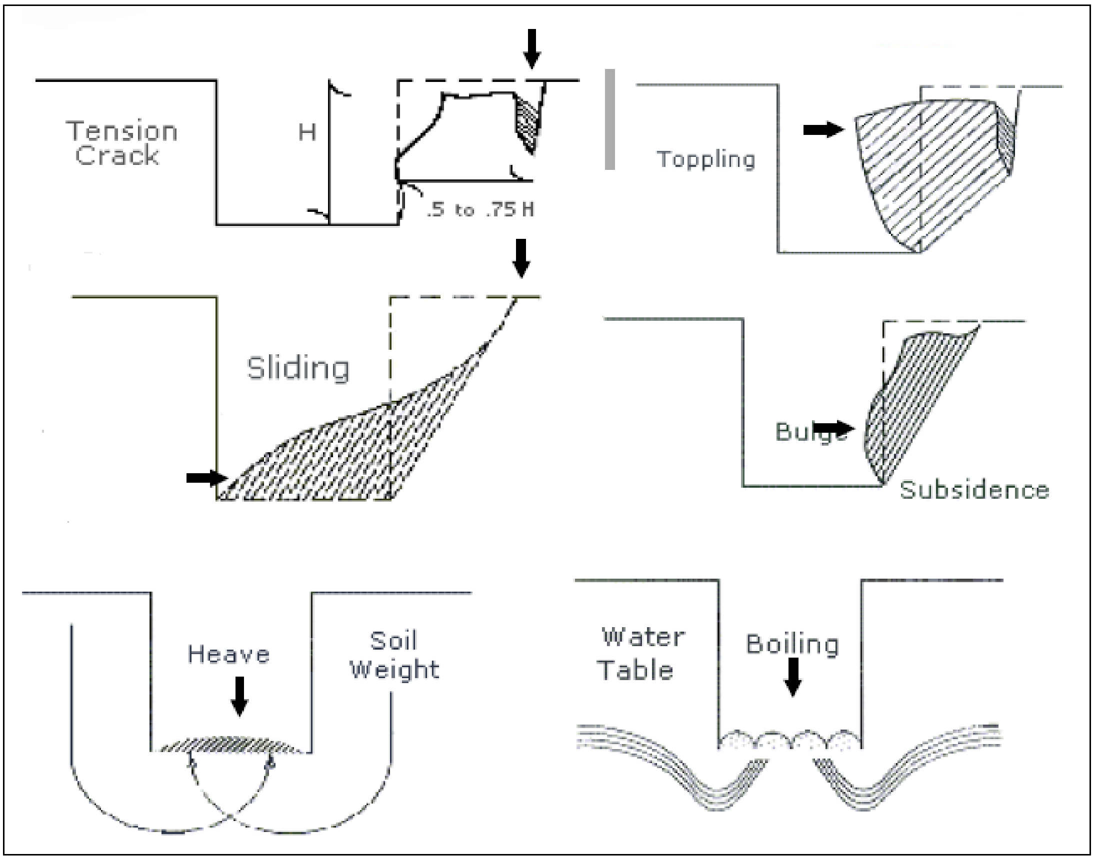

Figure 2.1: Different modes of failure in vertical cutting without any supports [1]

2.4 Supporting system for deep excavation

A support which is placed appropriately against the excavation vertical surface could prevent the soil lateral movement. Deep Excavation support systems are often temporary or permanent (depend on design) earth retaining structures that keep the sides of excavation to be vertical, stable, and ensure that movements will not cause damage to neighboring buildings, employees, public utilities in the surrounding ground, and general speaking stakeholders of the project.

2.5 First facade supporting system for deep excavation

As a first facade in supporting system alternatives, kind of cantilever retaining walls could be used. Retaining walls alone could be used as a cantilever beam or shell structure that the penetration length of the wall into the stiff layer supports it. Slurry diaphragm wall, pile wall-contiguous, pile wall-secant, pile wall-tangent, sheet pile wall, and soldier beam (with lagging) are kind of retaining walls used in deep excavation to prevent not only moving soil into the cut zone but also avert excess movement of retained ground in order to safety and operation of adjacent building or utilities. The penetration of the wall under the final depth of excavating especially in stiffer soil and also maximum allowable wall drift are important for providing cantilever condition of wall alone. A schematic cantilevered wall is shown in figure 2.2. Typical cross-sections of precast diaphragm wall, pile wall-contiguous, pile wall-secant hard/soft, pile wall-secant hard/hard, pile wall- tangent, sheet pile wall, and soldier beam with lagging as a wall are shown in figures 2.3, 2.4, 2.5, 2.6, 2.7, 2.8, 2.9 respectively.

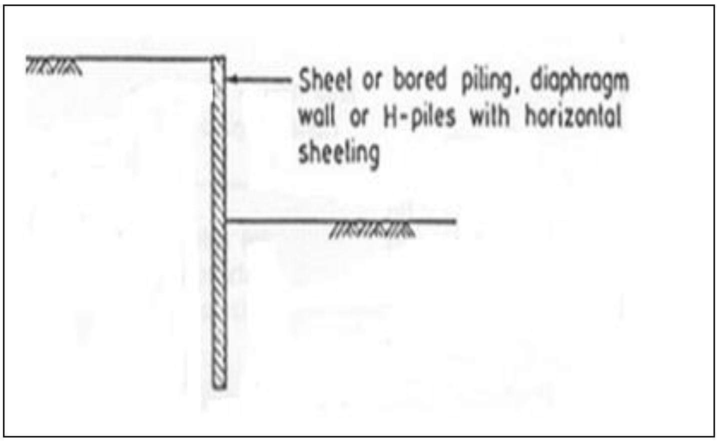

Figure 2.2: Schematic cantilevered wall cross-section [2]

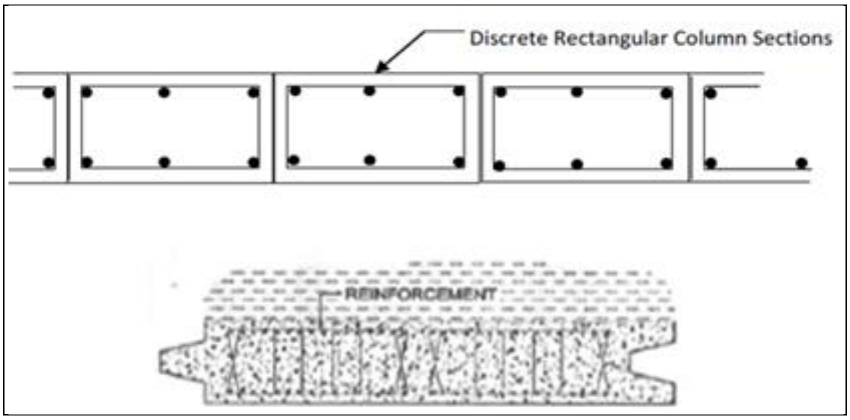

Figure 2.3: Schematically precast diaphragm wall cross-sections

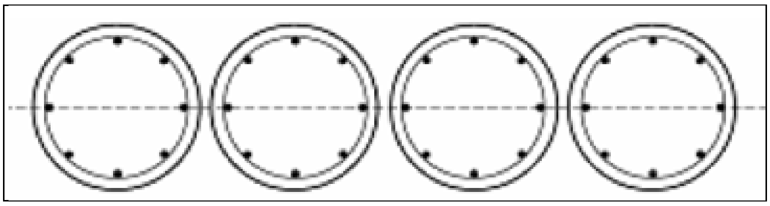
Figure 2.4: Pile wall-Contiguous cross-sections [11]

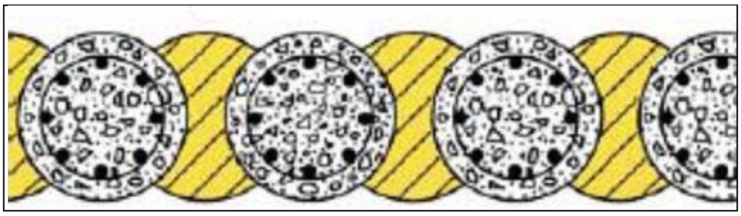
Figure 2.5: Pile wall-Secant hard/soft cross-sections [12]

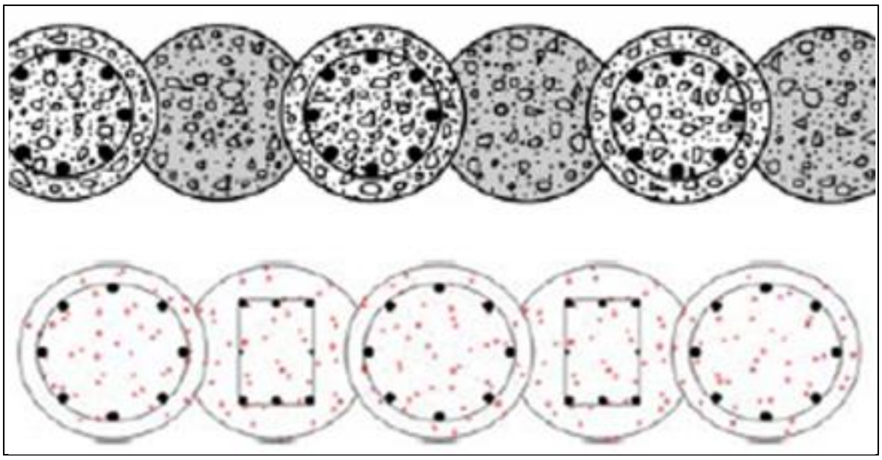
Figure 2.6: Pile wall-Secant hard/hard cross-sections [12]

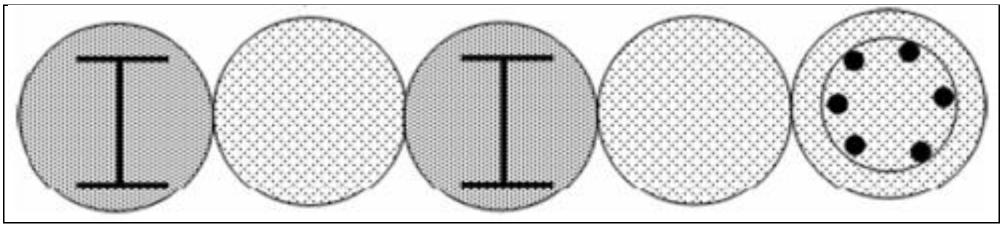

Figure 2.7: Pile wall-Tangent cross-sections [12]

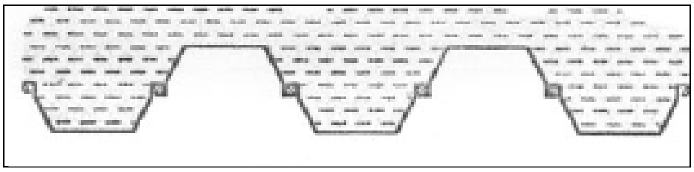

Figure 2.8: Sheet Pile wall cross-sections [2]

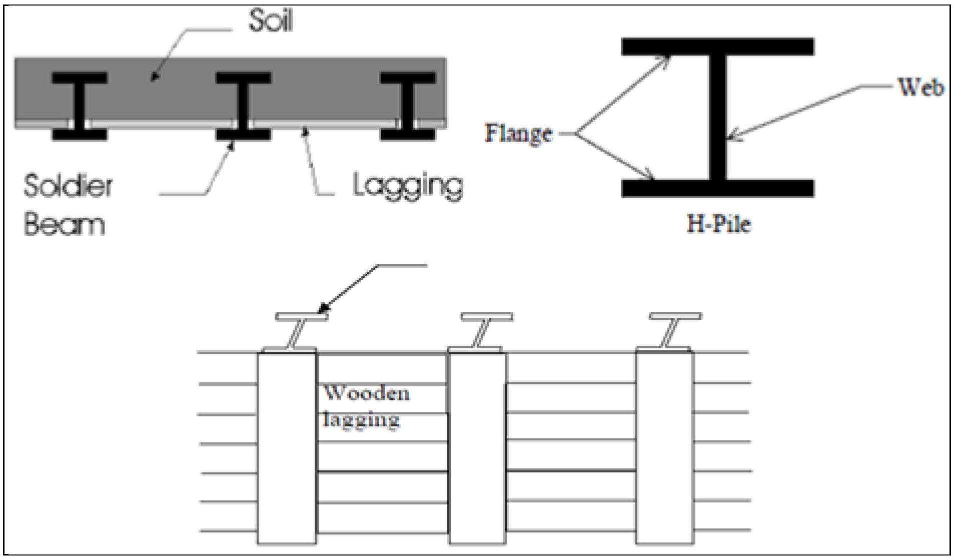

Figure 2.9: Soldier beam and lagging cross-section and view [10]

Piles could be placed in holes drilled into the underground or could be driven in soil or could be injected grouting [13] into the soil or rock by pressure. Drilling holes has a minimum noise and vibration in comparing driving [2]. Pile driving has strong vibration that can intensively affect to adjacent buildings. Steel sheet pile wall could be driven by drop hammer pile driver in soils within 50 or less standard penetration test (SPT) and suitable for lower deep excavation relatively[2]. Lower stiffness in relatively deeper cases, poor water-tightness (seepage is expected) in highly level of water table, and steel corrosion are disadvantages of sheet pile wall. Ability to get out after using at least case as waste and in optimistic case as intact is advantage for that in short term moderately. Soldier pile wall could be used in stiff soils for small and lower deep excavation temporarily. It is not suitable to use in soft clay, loose sand, and/or below ground water table (unless dewatering is done). Steel H-section column could be placed in holes drilled into the soil and grouting with weak concrete (not as rigid as other retaining systems) or could be driven in soil by drop hammer pile driver. Contiguous pile wall could be used in stiff soil and lower water table temporarily or permanently. It could be placed in holes drilled into the soil. Lower costs, speed, higher capacity to overcome obstructions are its advantages. Additional works to forming an acceptable surface of the wall, lack of water tightness, and close control of alignment are its disadvantages. Investigating of drilling condition is usually necessary. It can be used where ground water is not vulnerability or where grouting could be used to prevent outflow among the piles. Secant pile wall has same advantages of Contiguous pile wall. In addition sensitive and collapsible soils could be protected by it in short term. Secant pile walls are used to build barrier for the managing groundwater seepage and to reduce faction in feeble and soggy soils. The opening between piles is filled with an unreinforced cement/bentonite mix in hard/soft type and feeble concrete in the hard/firm case of wall. The case of hard/firm is constructed by installing the primary reinforced concrete piles and then cutting into the primary piles for the secondary piles. Diameters could be ranged from 500 mm to 1500 mm. The case of hard/hard is constructed with high strength concrete and may be reinforced. The Secondary piles are cut into the concrete primary piles using heavy duty piling rigs fitted to primary piles [2]. Tangent pile walls are formed by series of drilled shafts positioned such that the neighboring shafts stroke each other, hence the name tangent wall. Diaphragm wall is permanent wall with most effective water tightness. It is not suitable for highly collapsible soil during excavation. For construction of Slurry Diaphragm wall, a particular length of deep trench is excavated by clamshell accompanied with bentonite slurry filling for preventing collapse of vertical side of trench, then reinforcing steel cage is lifted and lowered into the trench and the trench is filled with concrete from the bottom up using tremie pipes. The bentonite slurry is displaced by concrete, pumped to storage and recycled.

2.5.1 Diaphragm wall construction

Diaphragm wall is generally reinforced concrete wall constructed in the ground using bentonite, cement bentonite, or polymer based slurries. The technique involves excavating a narrow trench that is kept full of slurry. Walls of thickness between 300 to 2000 mm may be formed in this way up to depths of 80 meters and also there are other sections except rectangular cross-section such as T cross-sectional beam [14]. In some situations there is need to more thickness of wall unless the secondary or multi propped supports are used to reduce the wall thickness. It could be used in conjunction with top-down method. Noise levels limited to engine noise only. The procedure could be realized in imaging as shown in figure 2.10. Diaphragm wall is constructed in sequence of separate panels with work continuity then distances constructed with work continuity. The work continuity is matching the below procedure with eight processes:

1- Fixing of Alignment

2- Guide wall Construction

3- Trenching

4- Trench Cleaning

5- Stop-ends fixing (pipes for Joint formation between panels)

6- Reinforcement Cage lowering

7- Placing of Concrete (tremie Pipes 8-10 inches, tremie head, and lifting head)

8- Withdrawal of Stop-ends

In trench cutting, bentonit is conducted from bentonit silo to mixer and is mixed with water as slurry, then is pumped to slurry tank by centrifugal pump. After that, slurry is pumped (pipe 3"- 4") to trench and due to cutting is mixed with excavated soil as cut mud. The cut mud is pumped (30 - 40 hp and 6-7 inch pipe) to desander which separates excavated soil and bentonite. The recycled bentonite is conducted to slurry tank and is used in trenching again and excavated soil is going out. Slurry and cut mud flow sequence in diaphragm wall construction is shown in figure 2.11. In trench cutting, chisel about 1.5 - 3.5 ton is used for breaking of rock if exist.

The deep trenching for diaphragm walls is done by grabs. The grabs based on operation could be divided into Rope suspended grabs, hydraulic (Kelly) grabs, and hybrid grabs which are mounted on a crane boom that in total are named Clamshell. A schematically crawler-mounted Rope operated grab is shown in figure 2.12.

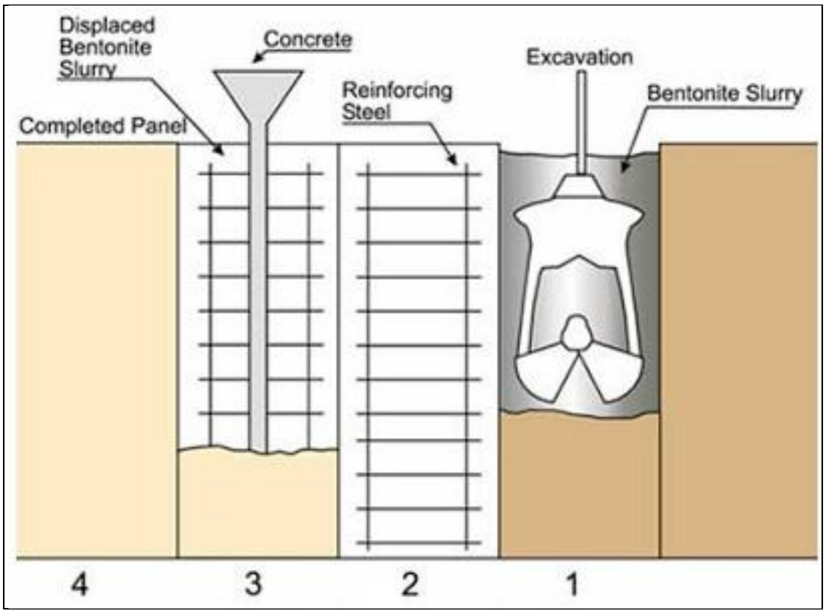

Figure 2.10: Diaphragm wall construction sequences in alternating panels (3-6 meters) [15]

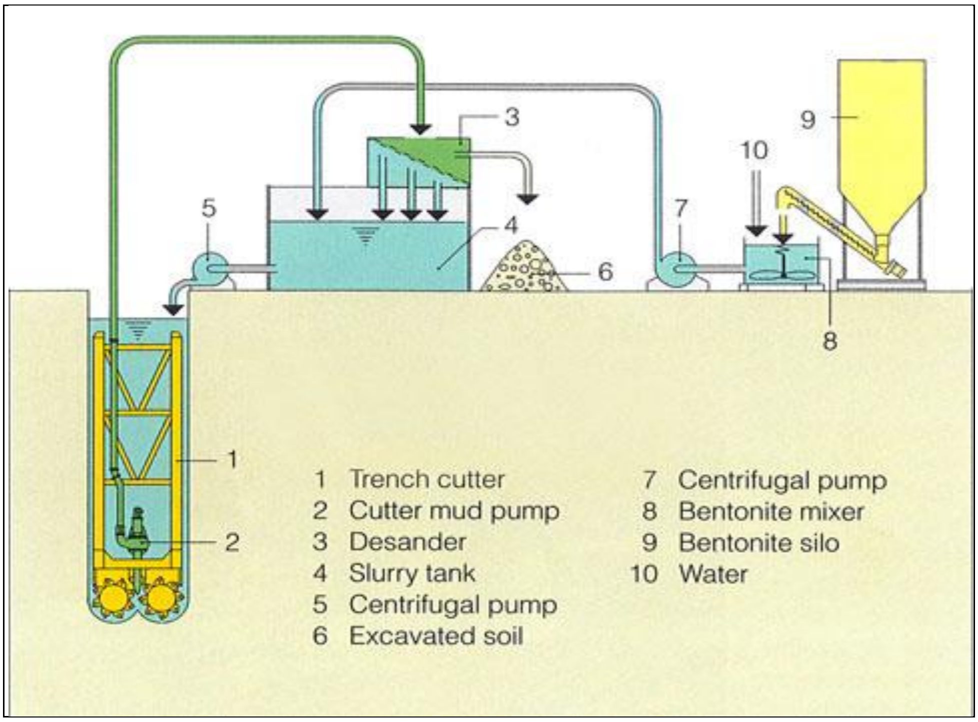

Figure 2.11: Slurry and cut mud flow sequence in diaphragm wall construction [15]

The grab is built-in with interlocking teeth to help the penetration to the soil. The efficiency of excavating is dependent on the self-weight of the attachment which limited to use in fairly loose soils and hydraulic system pressure if there is. It has ability of excavating to great vertical depth. However, it is difficult to control lateral digging action with rope operated grab. When other excavating methods are not possible the grabbing crane is selected. When long reaches such as deep trench excavation for underground diaphragm wall or over water is required, the grab could be used in conjunction with a long boom. The separate ropes are used for control the arrangement, opening and closing the shells, and change the position of the boom (figure 2.12). Hydraulically operated grab allows the grab to be opened and closed hydraulically and even rotated. Typically hydraulically operated grab is shown in figure 2.13 [16]. Expected Production of clamshell could be estimated by multiplication of volume per cycle, cycles per hour, grab fill factor, and job efficiency. Cycles per hour are function of hoisting speed, derricking, slewing speed, and travelling speed which are shown in table 2.1 as an approximate data for rope operated grab [17]. Volume per cycle is measured in term of heaped capacity. Clamshell bucket (grab) shapes is illustrated in figure 2.14 [10, 16]. An approximately comparison between empty weight and capacity of medium weight rope grabs (heaped capacity 15^{o} CECE- Committee on European construction equipment) and hydraulic grabs rating is shown in table 2.2 [16, 17] which shows weight differences vs. volume of grabs significantly. Typical range of properties for clamshell with hydraulic grab is shown in table 2.3 [18].

Figure 2.12: Crawler-mounted
ropes operated grab [18]

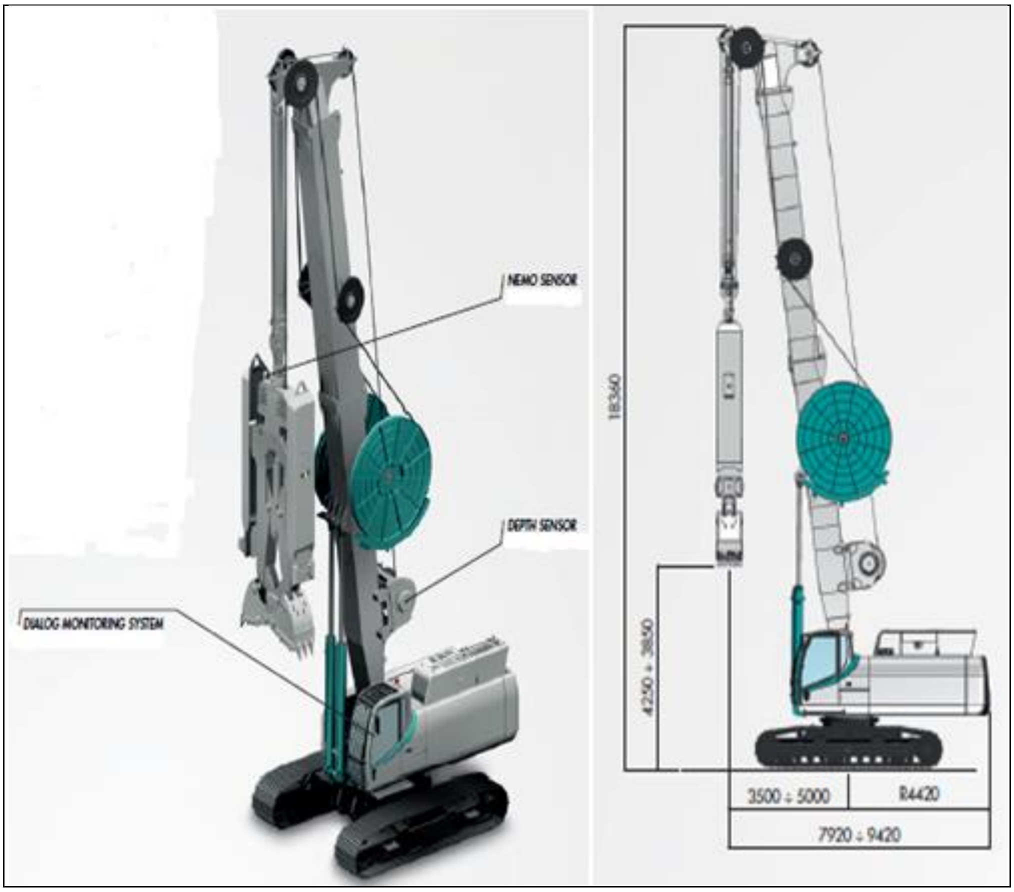

Figure 2.13: Crawler-mounted hydraulically operated grab [16]

Table 2.1: Crawler-mounted rope operated grab approximate data [17]

Hoisting speed	40 - 50 m/min
Derricking	50 - 100 sec
Slewing speed	2 rev/min
Travelling speed	1.5 - 3 km/hr
Maximum gradients when travelling	Loaded: 1 in 16 No load: 1 in 5

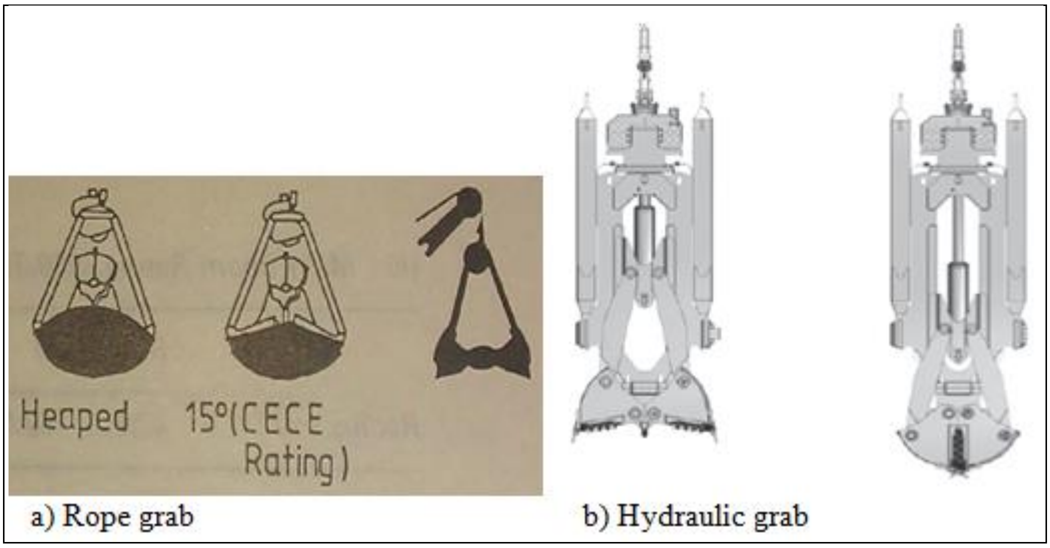

Figure 2.14: Clamshell bucket shapes: a) rope grab [17]   b) hydraulic grab [16]

Table 2.2: Approximately comparison between empty weight and capacity of medium weight rope grabs (heaped capacity 15^o CECE) and hydraulic grabs (HG) [16, 17]

Capacity	m^3	2.0- 2.5	1.75- 2.25	1.6- 2.	1.5-1.75	1.1- 1.4	1.0- 1.25
Empty weight	ton	2.4	2.35	1.8	1.75	1.37	1.35
Length	m	4.2	4.0	3.8	3.8	3.5	3.4
Capacity (HG)	m^3	3.04-3.48	1.77- 2.25	1.6-2.0	1.5-1.75	1- 1.5	0.63- 0.98
Empty weight (HG)	ton	20.5-21.2	17.3 -19.3	16-17.5	16 -17.5	15-16.4	13.6-14.8
Length (HG)	m	2.5-3.2	2.5-3.2	2.5-3.2	2.5-3.2	2.5-3.2	2.5-3.2

Table 2.3: Typical range of properties for large clamshell with hydraulic grabs [16]

Maximum Trenching depth (m)	50 - 80
Grab width (m)	0.5 – 1.50
Pull-down (kN)	360 - 460
Pull/push speed	35 - 115 m/min
Grab: opening time, closing time	5.5 - 9 sec, 6 -9 sec
Hydraulic system pressure	30 - 35 MPa
Engine power	230 - 300 kW
Work radius	4.65 – 5.35 m
Overall weight without grab	68 - 85 ton

2.5.2 Bored pile wall Construction

Bored pile wall is generally or one among reinforced concrete wall constructed in the underground. Noise levels limited to engine noise, casing driving noise, and vibration. Construction procedure of bored pile wall is:

1- Position of bored pile

2- Installation of casing

3- Drilling hole

4- Installation of cage (steel reinforcement) according to design into hole

5- Placing of concrete into well (tremie pipes 8-10 inches, tremie head, and lifting head)

6- Extraction of casing

The procedure could be realized in imaging for each pile as shown in figure 2.15 and for group of bored piles as a retaining wall in figure 2.16. Maximum deviation in horizontal position is 75 mm and maximum deviation in vertical position is 1 in 150 at any level. In practice piles should not bore next to other piles if the next pile is recently and less than 24 hours concreted or contains unset concrete. The side of the borehole in presence of water could be unstable and tend to collapse. In this case a temporary steel casing should be driven into stable stratum. The casing has normally about 30 mm thick and driven by vibro-hammer. The casing has to be driven for 5 – 6 numbers, and then excavating by auger method in soft clay and bucket method in stiff clay is done.

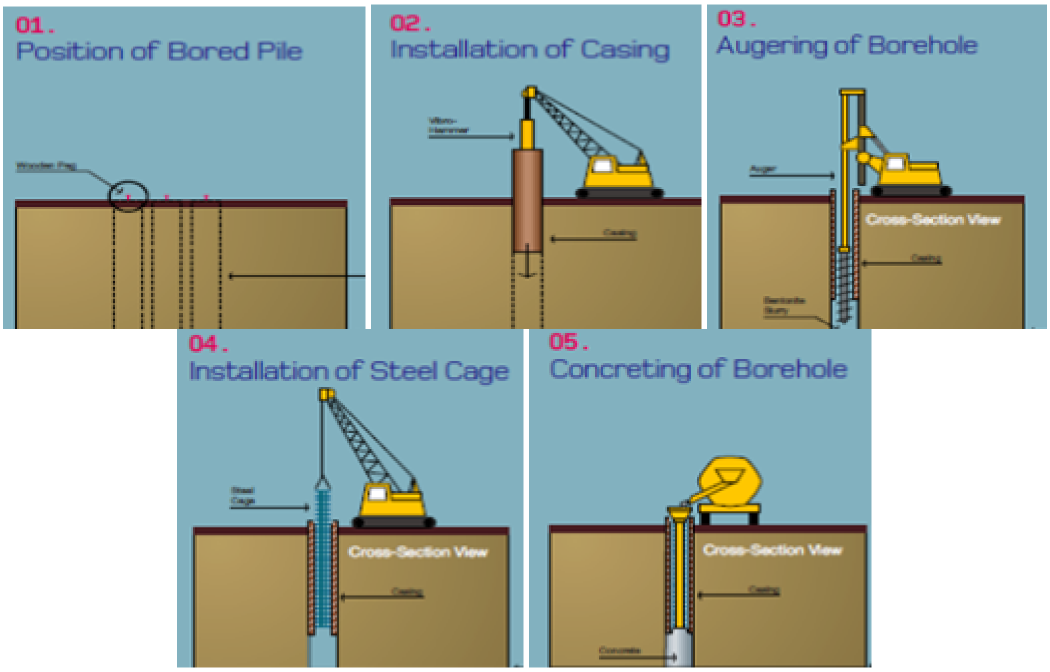

Figure 2.15: Procedure of a bored pile construction [11]

Figure 2.16: Procedure of sequential bored piles wall construction [11]

A hole is constructed by rotary drill rig or truck-mounted auger which is rotary drilling. The rotary drill rig includes of a major transporter, hoisting equipment as a support for cables and pulleys in and out of the hole, rotating equipment, and circulation equipment, to drilling (with a sharp rotates drill bit) a hole. Drill sits on a mast above the hole and the rotation of drill is gotten from a motor (e.g. electrics, hydraulics, or pneumatics). It has ability to cut through hardest underground. The rotary drill bit is located at the bottom of the drill string. With rotation of the drill, the hole becomes deeper and deeper and the drill string is reached to about 6 meters sections that they are joined together to help the pipe extend down the hole. The circulating system removes cuttings and debris, and coats the walls of the well with a mud-type coat to facilitate circulation. As the drill is gone down the hole and driven rotationally, the circulation equipment is cleaning the debris. There are three main types of drill bits: blade comprises steel or tungsten carbide, steel tooth rotary bit, and polycrystalline diamond bit (40 to 50 times harder than traditional steel bits) [19]. A crawler-mounted rotary drilling rig is shown in figure 2.17. Expected rotary drilling rig production is multiplication of volume per cycle and cycles per hour [20]. Typical crawler mounted bucket drill rig data up to 38 m drilling depth is indicated in table 2.4 [18].

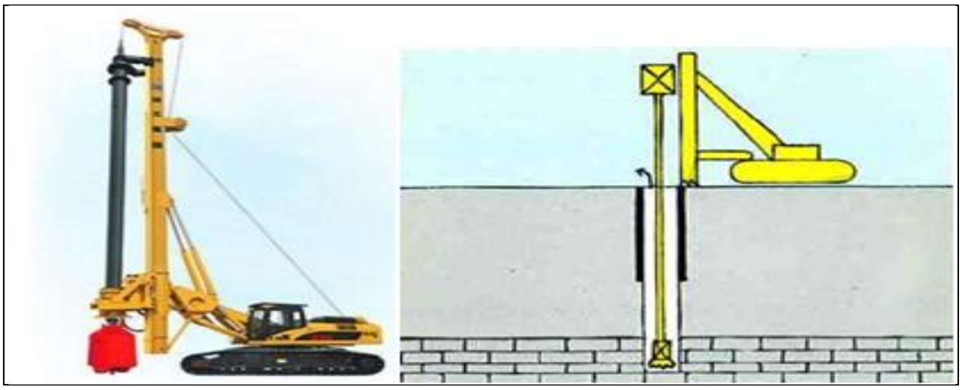

Figure 2.17: Crawler-mounted rotary bucket drilling rig with bucket [18]

Table 2.4: Typical crawler mounted bucket drill rig data up to 38 m drilling depth [18]

Maximum drilling depth	38 meters
Maximum drilling diameter	1.30 meters
Dimension in working condition (L×W×H)	7.122×4.2×17.25 m
Dimension in transportation condition (L×W×H)	12.3×3.2×3.06 m
Overall drilling weight (drilling tools not included)	45.5 tons
Engine rated power / rotary speed	133 kW/ 2000 r/min
Rotary speed	6 - 28 rpm
Main winch pulling force	145 kN
Main winch wire speed	75 m/min
Main winch Wire diameter	26 mm
Maximum complete device running speed	3 – 3.5 km/hr

2.5.3 Excavator

Hydraulic excavator-backhoes are commonly used for deep excavation in urban. The important construction operating factors in selecting of hydraulic excavator-backhoes in addition to cost and time scheduling, are flywheel power (hp or kW), operating weight, bucket heaped capacity (as is shown in figure 2.18), maximum digging depth (the C in figure 2.19), maximum loading height (the A in figure 2.19), and maximum reach at ground level (the B in figure 2.19). In-situ soils (bank cubic meter, BCM or yard, as BCY) which have been excavated (bulk) or loaded are in loose (loose cubic meter, LCM or yard, as LCY) state that volume is more than bank case and is swelled. Heaped volume is maximum loose volume of soil that could be placed in the bucket without spillages based on a specific angle of repose for the soil (25^o for dry sand, 32^o for dry common earth, 35^o for clay or gravel, and 37^o for moist common earth or moist sand [20]) in the bucket. Volume of soil in bucket is estimated by multiplication of nominal bucket volume and bucket fill factor (table 2.8 [20]). Approximate range of constructional characteristics of hydraulic backhoe-excavator machines are classified from different references that indicated in table 2.5 [17, 21, 22, 23, 24, 25]. Bucket heaped capacity has main role in excavating production if the other requirements of machine are the same or near. This is because of the dependence of filled bucket weight to power of machine and flywheel, boom force and length, chassis, and footing of machine on ground that can be track or wheels. Advantages and disadvantages of tracks and wheels for using as footing of excavator are crucial in excavator selecting.

Tracks advantages are maneuverability, sever underfoot, traction, flotation, and relatively lower pressure on working ground while wheels advantages are mobility and speed, better stability with outriggers or dozer, without any pavement damage, and dozing capability. Tracks disadvantages are road pavement damage while wheels advantages are leveling with repositioning outriggers, and relatively higher pressure on working ground.

The production of hydraulic backhoes could be estimated by [20, 26]:

Expected Production (Lm^3) = $C \times S \times V \times B \times E$ (Eq 1.1)

Where,

C = cycles/hour (table 2.6 [20]),

S = swing depth factor (table 2.7 [20]),

V= heaped bucket volume (Lm^3),

B = bucket fill factor (table 2.8 [20]),

E = job efficiency (ratio of work in hour).

Cycle time estimating chart for CAT productions based on bucket size, and soil type is shown in figure 2.20 [25]. Total cycle time includes: load bucket, swing loaded, dump bucket, and swing empty.

The job efficiency is ratio of work in hour. There are two basically approach to estimating job efficiency [20]. One is to use the number of effective working minutes per hour that in other word is the working minutes divided by 60 minutes. In that case weather condition (about 10%), maneuvering (approximately 8%), mechanical breakdowns (about 5%), operator efficiency (nearly 7%), and waiting for dump trucks (approximately 10%) could influence the job efficiency [17] which could reach to 45%. A proper plan of excavation path in a site can improve the efficiency of maneuvering, trucks way and position which typically is shown in figure 2.21 [27]. The other approach for estimating job efficiency is multiply the number of theoretical cycles per 60 minutes by a numerical efficiency factor from table 2.10 [20] that gives job efficiency factors based on management condition vs. job condition.

In-situ volume of ground soil is surveyed for excavation while the volume of excavated soil and production is based on swelled soil. The range of excavation swell factor for four kinds of soils is illustrated in table 2.9 for converting and comparing [17, 20]. It is proposed for each work and each soil layer the swell factor is detected experimentally.

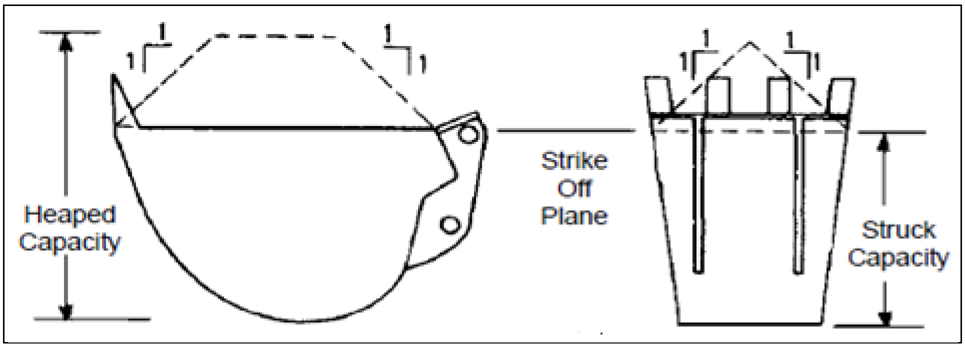

Figure 2.18: Excavator heaped and struck capacity volume concepts in bucket rating [25]

Table 2.5: Approximate range of constructional characteristics for hydraulic backhoe excavator machines [17, 21, 22, 23, 24, 25].

Flywheel power (hp)	**Operating weight** (ton metric)	**Bucket heaped capacity** (m^3)	**Maximum digging depth** (m)	**Maximum loading height** (m)	**Maximum reach at ground level** (m)
15-20	1.6 -1.7	0.018- 0.06	1.8 -2.13	2.36	3.4 -3.7
50-60	6.5 -7.6	0.14 - 0.28	4.1 -5.59	4.16 -5.57	6.2-7.42
70 -90	11.1 -13.27	0.24 - 0.78	4.14 - 6.27	5.3 -7.57	7.29 -9.22
95 -110	15.8 -16.4	0.35 - 0.90	4.8 - 6.27	5.83 - 8.04	7.79 -9.53
114 -124	13.81-18.41	0.24 -1.35	4.42 -7.44	5.6 -5.82	8.27 -10.55
128 -140	17.91-22.53	0.4 -1.5	6.14 -7.68	6.06 - 8.07	8.65-10.96
153 -168	22.76-28.59	0.45 -2.2	5.4 -7.22	5.89 - 8.59	8.89 -10.57
200 -290	32.42- 49	0.66 -3.5	6.1- 9.57	6.25 - 8.45	9.64 -13.45
428	75.77- 80.7	1.5 -5.6	6.94 -10.84	8.26-10.35	12.0 -15.96
800	182	8.5 -18.3	8.4 - 9.7	9.1- 9.7	14.9 -16.1
1470	316.6	13 -27.5	9.4	9.8	17.7

Table 2.6: Standard cycles per hour for hydraulic backhoes (machine size vs. type of ground soil) [20, 26]

Type of material	**Small bucket excavator** (0.76 Lm^3 or less)	**Medium bucket excavator** (0.94 to 1.72 Lm^3)	**Large bucket excavator** (1.72 Lm^3 & more)
Soft(sand, gravel, loam)	250	200	150
Average common earth, soft clay)	200	160	120
Hard (tough clay, rock)	160	130	100

Table 2.7: Swing-depth factor for backhoes (angle of swing in degree vs. % optimum depth of cut) [26, 28]

Angle of swing in degree	**45°**	**60°**	**75°**	**90°**	**120°**	**180°**
30% Optimum depth of cut	1.33	1.26	1.21	1.15	1.08	0.95
50% Optimum depth of cut	1.28	1.21	1.16	1.10	1.03	0.91
70% Optimum depth of cut	1.16	1.10	1.05	1.00	0.94	0.83
90% Optimum depth of cut	1.04	1.00	0.95	0.90	0.85	0.75

Table 2.8: Bucket fill factors for excavators [26, 28]

Material	**Bucket fill factor**
Common earth, loam	0.80 -1.10
Sand & gravel	0.90 -1.00
Hard clay	0.65 – 0.95
Wet clay	0.50 – 0.90
Rock, well blasted	0.70 – 0.90
Rock, poorly blasted	0.40 – 0.70

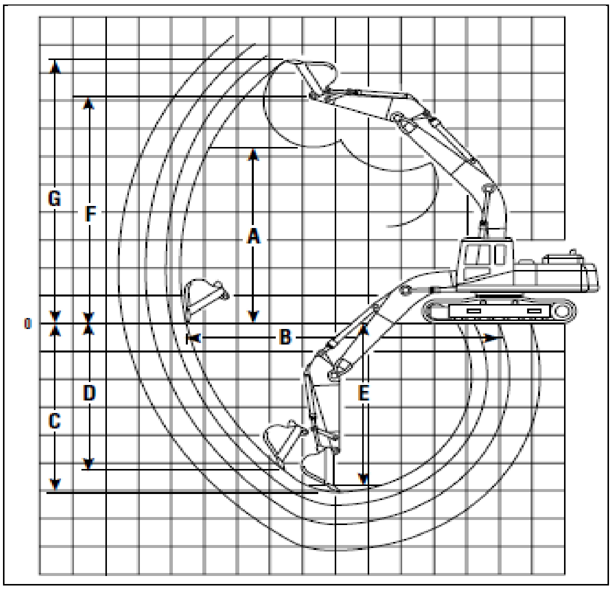

Figure 2.19: General scheme of hydraulic backhoe-excavator working potential [25]

Model	307	311B	312B, 312B L	315B, 315B L	318B L*, 318B LN	320B	322B	325B	330B	345B*	350	375	5130 ME	5230 ME
Bucket Size L (yd^3)	280 0.37	450 0.59	520 0.68	520 0.68		800 1.05	1000 1.31	1100 1.44	1400 1.83		1900 2.5	2800 3.66	10 m^3 13	15.5 m^3 20.3
Soil Type	Packed Earth					Hard Clay								
Digging Depth (m) (ft)	1.5 5	1.5 5	1.8 6	3.0 10		2.3 8	3.2 10	3.2 10	3.4 11		4.2 14	5.2 17	4.0 13	5.0 16
Load Bucket (min)	0.08	0.07	0.07	0.10		0.09	0.09	0.09	0.09		0.10	0.11	0.12	0.12
Swing Loaded (min)	0.05	0.06	0.06	0.04		0.06	0.06	0.06	0.07		0.09	0.10	0.13	0.14
Dump Bucket (min)	0.03	0.03	0.03	0.02		0.03	0.04	0.04	0.04		0.04	0.04	0.04	0.04
Swing Empty (min)	0.06	0.05	0.05	0.05		0.05	0.06	0.06	0.07		0.07	0.09	0.13	0.14
Total Cycle Time (min)	0.22	0.21	0.21	0.21		0.23	0.25	0.25	0.27		0.30	0.34	0.42	0.44

Figure 2.20: Chart for estimating cycle time of CAT backhoe productions [25]

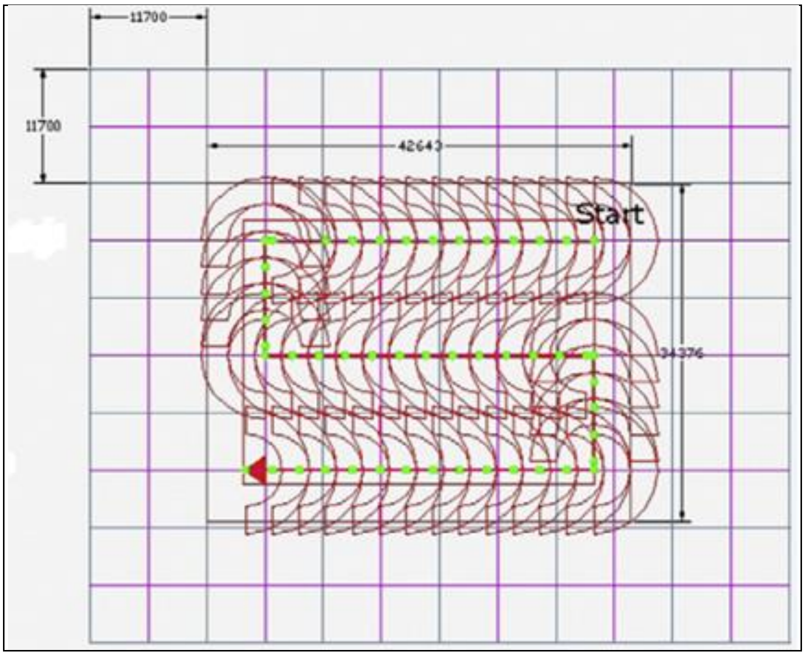

Figure 2.21: Typical excavation paths in a site [27]

Table 2.9: The range of excavation swell factor [20, 26]

Ground Soil type	Swell factor
Common earth	1.1 – 1.3
Sand	1.0 – 1.3
Clay	1.25 – 1.4
Gravel	1.0 – 1.12
Broken rock	1.5 – 2.0

Table 2.10: Job efficiency factors based on management condition vs. job condition [20]

Job condition	**Management condition: Excellent**	**Management condition: Good**	**Management condition: Fair**	**Management condition: Poor**
Excellent	0.84	0.81	0.76	0.70
Good	0.78	0.75	0.71	0.65
Fair	0.72	0.69	0.65	0.60
Poor	0.63	0.61	0.57	0.52

2.6 Second facade supporting system for deep excavation

If each of the mentioned walls and their cases or conditions is not enough alone to prevent the soil lateral movement, or there is not space for required wall thickness then there is need to second facade in one level for improving the supporting system. In this case temporary strutting (wood, steel, or reinforced concrete beams or frames) system front the wall and/or anchorage (ordinary anchorage and tie-back) as internal bracing system behind the wall and/or permanent slab with relevant framing (top-bottom method with pile) in one level (just one row) could be used as secondary supporting systems after implementing wall. In this case there could be two stages for excavation. A schematic strutted cantilevered wall in relatively narrow excavation and a schematic strutted cantilevered wall in relatively wider excavation are shown in figures 2.22, 2.23 and a schematic anchored cantilevered wall is shown in figure 2.24. Typical section of anchored piles wall contiguous and section of anchored piles wall secant are shown in figures 2.25, 2.26.

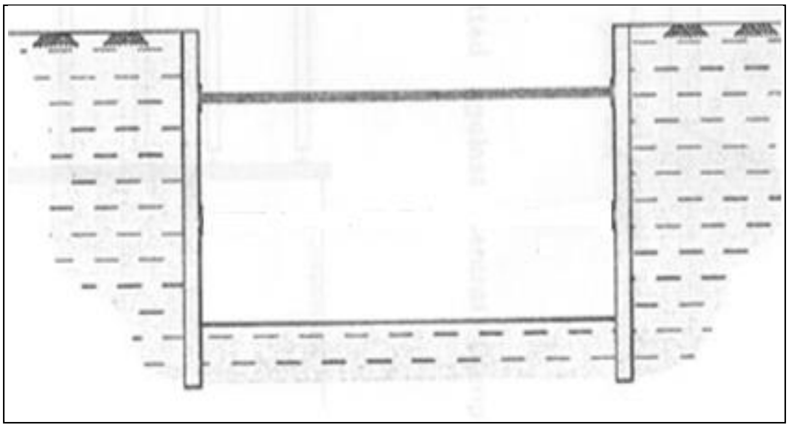

Figure 2.22: Schematic braced cantilever wall in narrow excavation [2]

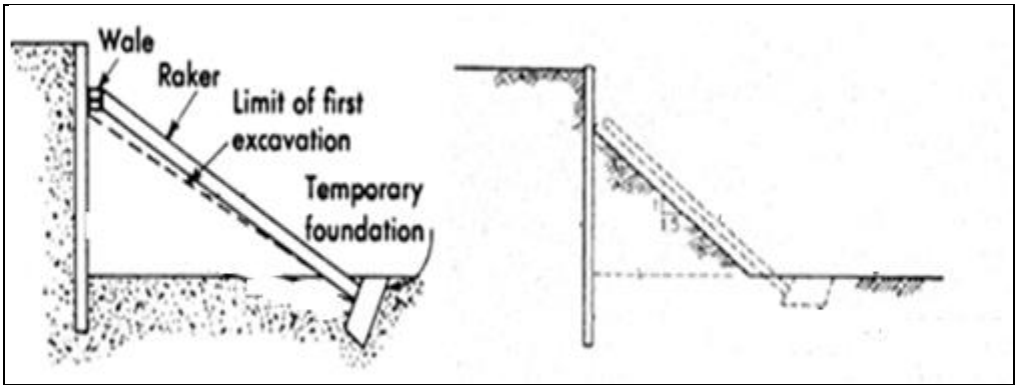

Figure 2.23: Schematic braced (rakers) cantilever wall in long span excavation [10]

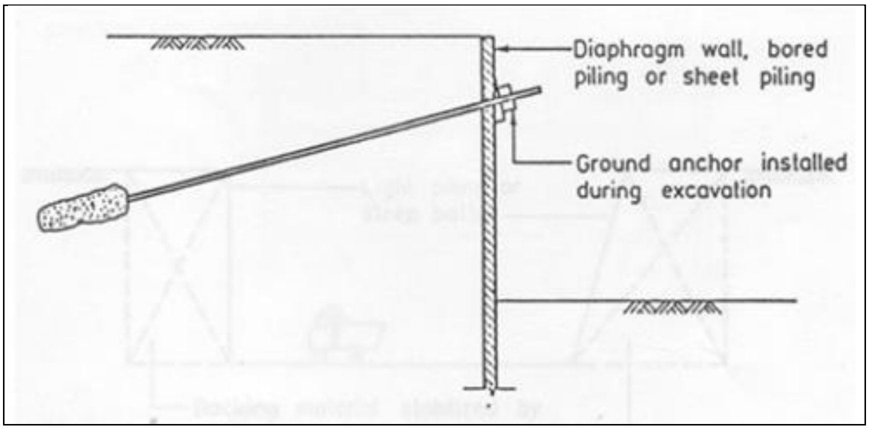

Figure 2.24: Schematic anchored wall [2]

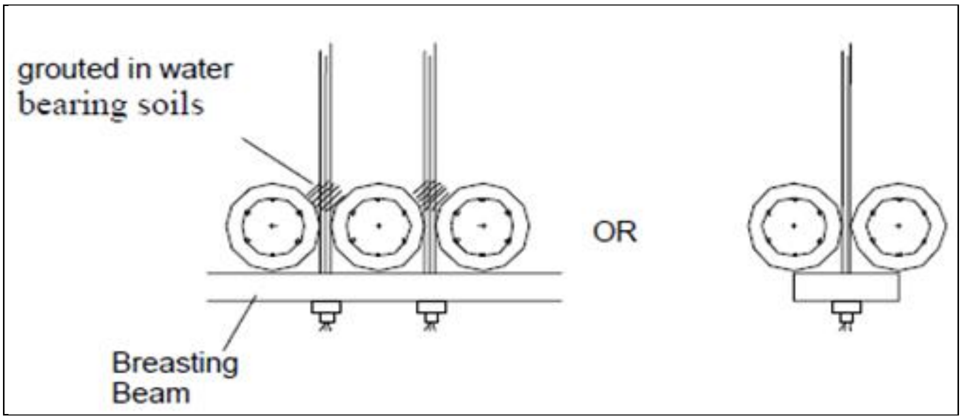

Figure 2.25: Typical section of anchored contiguous Pile wall [2]

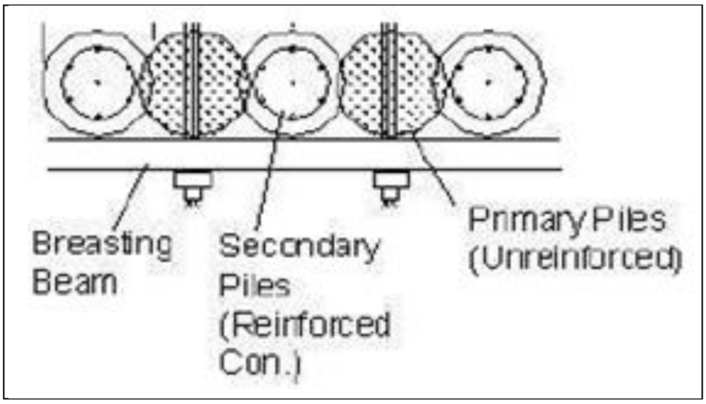

Figure 2.26: Typical section of anchored secant pile wall [2]

2.6.1 Anchor, tie back, and soil nailing

An anchor implementation process is:

1- Drilling a hole with an auger (100-150 mm diameter and specific length according to design)

2- Placing strand or bar in the hole

3- Injecting concrete grouting under pressure in a hole along the anchor length

4- Minimum 10 days waiting period to reach the minimum 30 MPa strength for grout

5- Check to achieve full cohesion and friction of anchor with soil

6- Made the connection with wall

A pre-stressed anchor is named tie back. The vertical area on wall that an anchor could hold is about 2.5 to 12 square meters that the holding area is depend to kind of soil, anchor size, anchor material characteristic strength, water table, and pressure. For example, reinforced concrete wall with 5760 square meters (maximum 19.2 meters excavation depth) had been shored by 500 tie backs [10]. High tensile strength steel bars in lengths up to 25 meters in sizes from 18 mm to 50 mm diameter with yielding/ultimate strength as a characteristic strength between 835/1030 MPa and 1080/1230 MPa are available in market commonly [3]. Due to creep, the allowable pre-stressing is assumed about 60% of the yielding strength or failing load [29]. Also there are high strength steel strands with seven wires that each wire has 12.5, 15, or 18 mm diameter, and 1600/1900 MPa yielding/ultimate characteristic strength [17].

Working Pressure, water cement ratio (w/c) and additives of grouting depend on the permeability and stiffness of the soil. Grout could fracture or push the soil around depending on type of soil, grout and pressure level. Grouting could be in low pressure (less than one MPa) or high pressure (more than two MPa) to transfer bond stress. Granular and alluvial soils and weak rocks are generally grouted with more than a few bars of pressure through casing or using packer. Stiff cohesive soils and silts may be grouted at higher pressures (greater than 15-20 bars) [3]. Capacity could be altered between 350 kN to 1400 kN from fine sand to dense sand and gravel.

In situ reinforcing of the soil while it is excavated from the top down is soil nailing. Typical construction sequence begins with the excavation of a shallow cut. Then shotcrete is applied to the face of the cut and soil nails are drilled and grouted [30].

Rapid and economical, the least troublesome method to construct a retaining wall (not cantilever), and requiring an unusual amount of hand work, craftsmanship and geotechnical knowledge to construct are soil nailing advantages.

The measured pull-out capacity of the soil nails in compacted sand under unsaturated conditions was found to be 1.3 to 1.7 times higher than the pull-out capacity under saturated conditions [31]. Measured Pull out capacity for a hole with 100 mm diameter included 22 mm diameter threaded bar grouted nail was from 1.98 KN to 3.42 KN per 0.8 meter [31]. The water cement ratio of grout was 0.45 and the yield stress of bar was 517 MPa [31].

Excessive movement of adjacent buildings was caused by Nailing as disadvantages such as 7.6 mm settlement at the face of the wall, and 2.5 mm at 10.97 meters behind the wall in Excavation depth of 12.20 meters [2, 4]. Movement of adjacent buildings is caused by nailing even in quasi-benching excavation with the belowest section width of 22.5 meters in excavation depth of 30 meters and facilities was moved about 50 mm at the face of the wall by that excavation.

The free anchor length is the distance between the anchor head and the proximal end of the grout. The fixed anchor length is the length of anchorage which the tensile load is capable of being transmitted to the surrounding ground. The fixed anchor length shall not be less than 3 meters or specific fixed length for all anchors.

Anchor support is rapid, and economic. The need to long term maintenance in permanent ground anchor, problems due to removing after using, adjacent owner disagreement, leakage and loss of fine through drill holes, and realized movement of adjacent foundation [32] are its disadvantages. Typical properties of horizontal drills, grouting and jet-grouting pumps are indicated in tables 2.11, 2.12 and 2.13 respectively.

Table 2.11: Typical properties of crawler horizontal drills [28]

Engine power (kW)	**74**	**85**	**95**
Mast stroke of rotary head (mm)	2350	4000-5000	4000-6700
Mast extraction force (kN)	45	50	85
Mast crowd force (kN)	45	50	85
Clamps diameter (mm)	40 - 254	40 - 254	40 - 254
Rotary head drilling speed (rpm)	56 - 112	56 - 112	52 - 400
Rotary head torque (Nm)	7200	10200	15200

Table 2.12: Typical properties of grouting pumps [5]

Engine power (kW)	**45**	**0.65**	**0.3 - 20**
Maximum grout pressure (Bar)	110	400	10 - 60
Maximum flow rate (liter/min)	100 -115	1	1-200
High pressure output diameter (mm)	25	25	25

Table 2.13: Typical properties of jet-grouting pumps [16]

Engine power (kW)	317	373	440	522
Maximum grout pressure (Bar)	800	800	800	900
Maximum flow rate (lit/min)	480	625	675	875
High pressure output diameter (mm)	38	38	50	50

2.6.2 Strutting

Strutting are temporary horizontal layer of elements or frame of steel, wood, or reinforced concrete beam-column elements front the wall so that wall-strut connections (wale) strength and displacement, and buckling of beam-column elements are important factors in stability of supporting system which can be learnt by case histories [1]. In narrow wide of excavation, raker brace are used (figure 2.23). Struts is bought newly and after work convert to waste ordinarily.

2.7 Multi-level secondary supports with multi stage excavation

In case of deeper vertical cutting or special complex situations, there is need to more bracing (multi-propped wall) or more anchors (multi-tied walls) in different row levels simultaneously with staged-cutting to keep the required safety to reach to final depth of excavation. For example, an excavation processes in clay is shown in figure 2.27 [33]. After implementing retaining wall, excavation is execute to -2 m level then at level -1 m wale and struts (bracing) is executed, then excavation is execute to -5 m level then at level -4 m wale and steel struts is executed, next excavation is execute to -8 m level then at level -7 m wale and steel struts is executed, and finally excavation is execute to final depth at -10 m level. If the depth of excavation becomes more, the stage of sectional excavation and adding supports is iterated to reach to final excavation depth. For other examples, four collected cases with their excavation stages, and construction methods after implementing retaining wall are indicated in tables 2.13 and 2.14 [6].

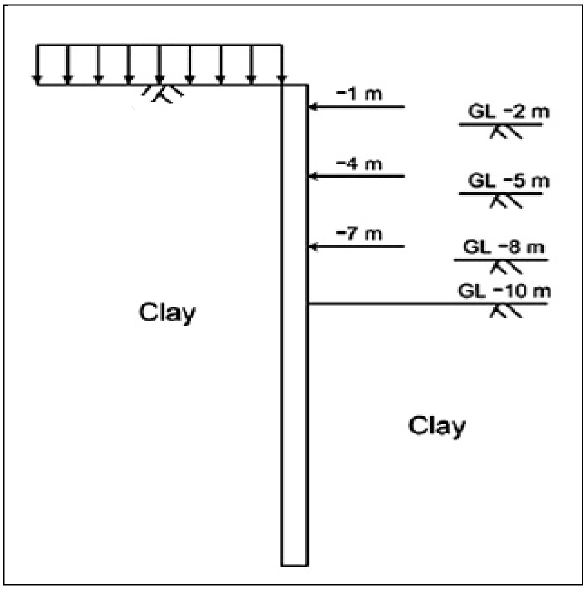

Figure 2.27: A typical deep excavation processes with strutted wall supporting system in clay [33]

Table 2.14: Four collected cases of deep excavation stages, and construction methods [6]

No	**Cases**	**Total excav. stages**	**Excav. width** (m)	**Final depth of excav**. (m)	**Wall length** (m)	**Wall thickness** (m)	**Construct method**
1	TNEC	7	41.20	19.70	35.0	0.90	Top-down
2	Formosa	7	33.40	18.45	31.0	0.70	Bottom-up
3	Far-Eastern	6	70.0	20.0	32.5	0.70	Top-down
4	Electronics	5	36.0	13.70	28.50	0.70	Bottom-up

Table 2.15: Four collected cases of deep excavation stages vs. levels of excavation and struts in meters unit underground [6]

Excavation stage number	Case No: 1 TNEC	Case No: 2 Formosa	Case No: 3 Far-Eastern	Case No: 4 Electronics
1	h_{excav} = 2.8 h_{slab} = ---	h_{excav} = 1.6 h_{strut} = ---	h_{excav} = 4.95 h_{slab} = ---	h_{excav} = 2.10 h_{strut} = ---
2	h_{excav} = 4.9 h_{slab} = 2.0	h_{excav} = 4.3 h_{strut} = 1.0	h_{excav} = 8.55 h_{slab} = 3.45	h_{excav} = 3.80 h_{strut} = 1.30
3	h_{excav} = 8.6 h_{slab} = 3.5 & 0	h_{excav} = 6.90 h_{strut} = 3.70	h_{excav}= 7.05 h_{slab} = 2.40	h_{excav} = 7.0 h_{strut} = 3.30
4	h_{excav} = 11.8 h_{slab} = 7.1	h_{excav} = 10.15 h_{strut} = 6.20	h_{excav} =10.90 h_{slab} = 5.40	h_{excav} = 11.10 h_{strut} = 6.50
5	h_{excav} = 15.2 h_{slab} = 10.3	h_{excav} = 13.20 h_{strut} = 9.50	h_{excav} =13.90 h_{slab} = 6.90	h_{excav} = 10.50 h_{strut} = 3.70
6	h_{excav} = 17.3 h_{slab} = 13.7	h_{excav} = 16.20 h_{strut} = 12.50	h_{excav} = 20.0 h_{slab} = 16.4	----- -----
7	h_{excav} = 19.7 h_{slab} = 16.5	h_{excav} = 18.45 h_{strut} = 15.5	------ ------	------ ------

In top-down method, floor slabs, structural frame and struts are used as support in lieu of struts or anchors from top to bottom. In bottom-top method, struts or anchors are used as support. As a schematically top-down method, it could be implemented according to processes shown in figures 2.28, and 2.29.

In figure 2.28 firstly the retaining wall in the edges is implemented. Then by drilling to under main foundation and implementing foundation of relatively out of central area, the columns that have plates for supporting floor slabs is installed on the foundations. Next the first basement floor level walling slab is implemented between the columns and retaining wall. Then the central area is excavated to under main foundation of central area so that the edge slopes of underground stayed stable (with 45 degree or less in normal soils). Next the foundation, structure, and slabs of central area is implemented so that its first basement floor level walling slab is jointed to edge corresponding pre-constructed slab. Then the edge slopes of underground is excavated as in stages (for levels number 1, 2, 3, and 4 in figure) so that in each stage the under stage slabs of central area is constructed and jointed to corresponding edge slabs in that levels until the excavation is finished and the main foundation of central area is jointed to edges foundation. Finally the superstructure is constructed.

In figure 2.29 firstly the retaining wall in the edges is implemented. Then by drilling with steel casing installing to more than under main foundation and implementing small temporary base of relatively out of central area, the steel lattice columns that have plates for supporting floor slabs is installed on the temporary foundations and walling beam at first basement floor level is installed between the wall and steel lattice columns. Next the excavation and walling slabs in downward stages (for levels number 1, 2, 3, 4, and 5 in figure) are implemented in the relatively out of central area. Then excavation of stage 6 and raft foundation implementing is done above small temporary base so that the lattice columns loads transmitted onto raft foundation. Finally the main structure and slabs are implemented in central area so that the slabs are jointed corresponding edge slabs.

Simultaneously implementing of two level of main structural frame as a multi level secondary support of permanent retaining wall with temporary raker struts is shown in figure 2.30. As we see the final stage is excavation of edge ground front of retaining wall.

A schematically plan and section of bottom-top method is shown in figure 2.31. In this case long flying shores at required levels depend on each excavation stage levels are implemented. In this case vertical supports to shore for long spans are required that for each of them there is need to small temporary base.

In relatively wider excavation area, and common soils, the typical processes and details of figure 2.32 which uses soldier beam as a truss with lagging could be implemented. The excavation and implementing of soldier beams is shown in eight stages and steel cross-sections with welding connections details is appropriate for about 10 meters excavation. It is possible the use of a foundation alone or with other retaining wall to support the retaining wall that are shown in figure 2.33 and figure 2.34. Permanent wall and two level temporary rakers for implementing main frame and slabs as a supporting system is shown in figure 2.35.

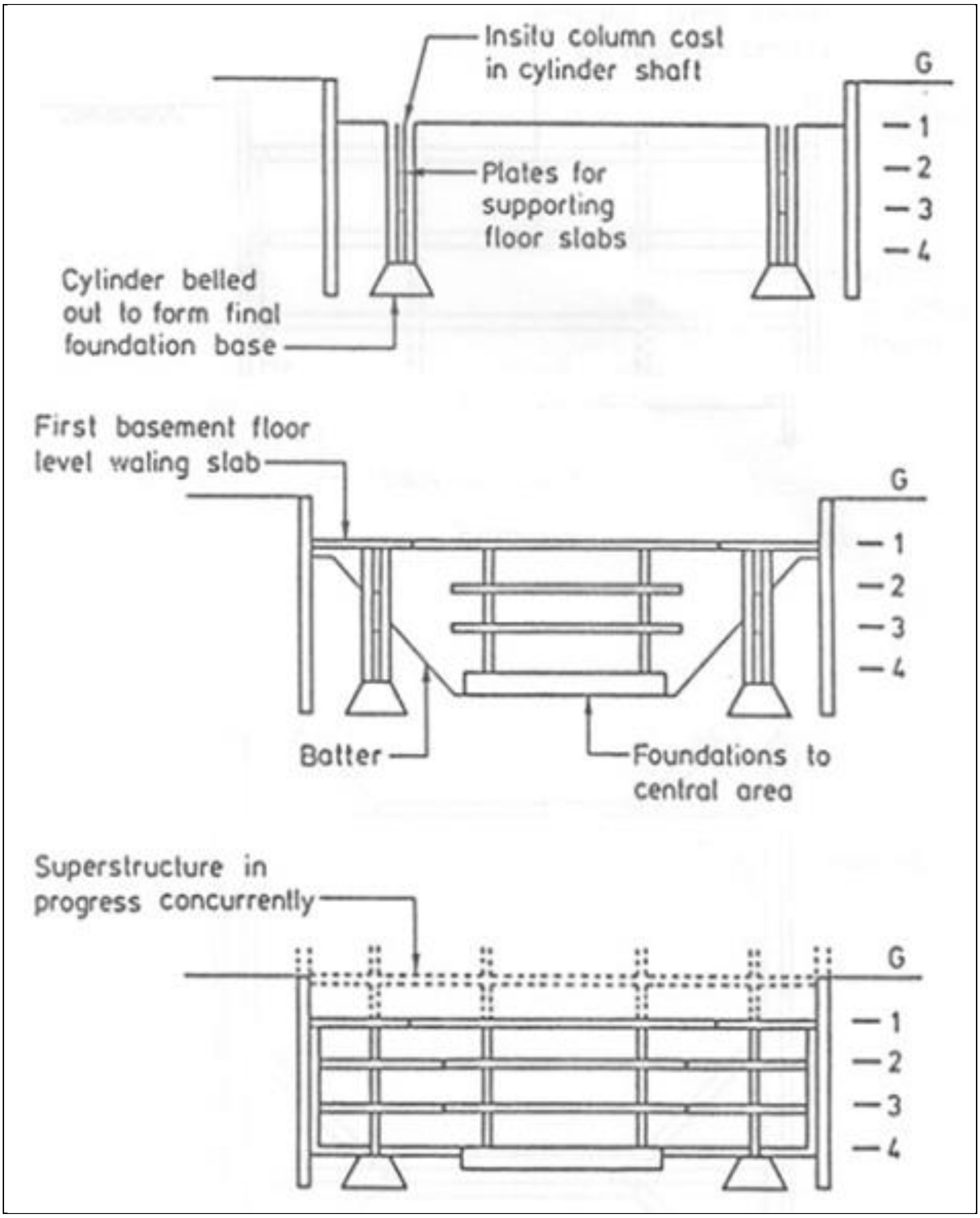

Figure 2.28: Typical processes for implementing top-bottom method - 4 basements [2]

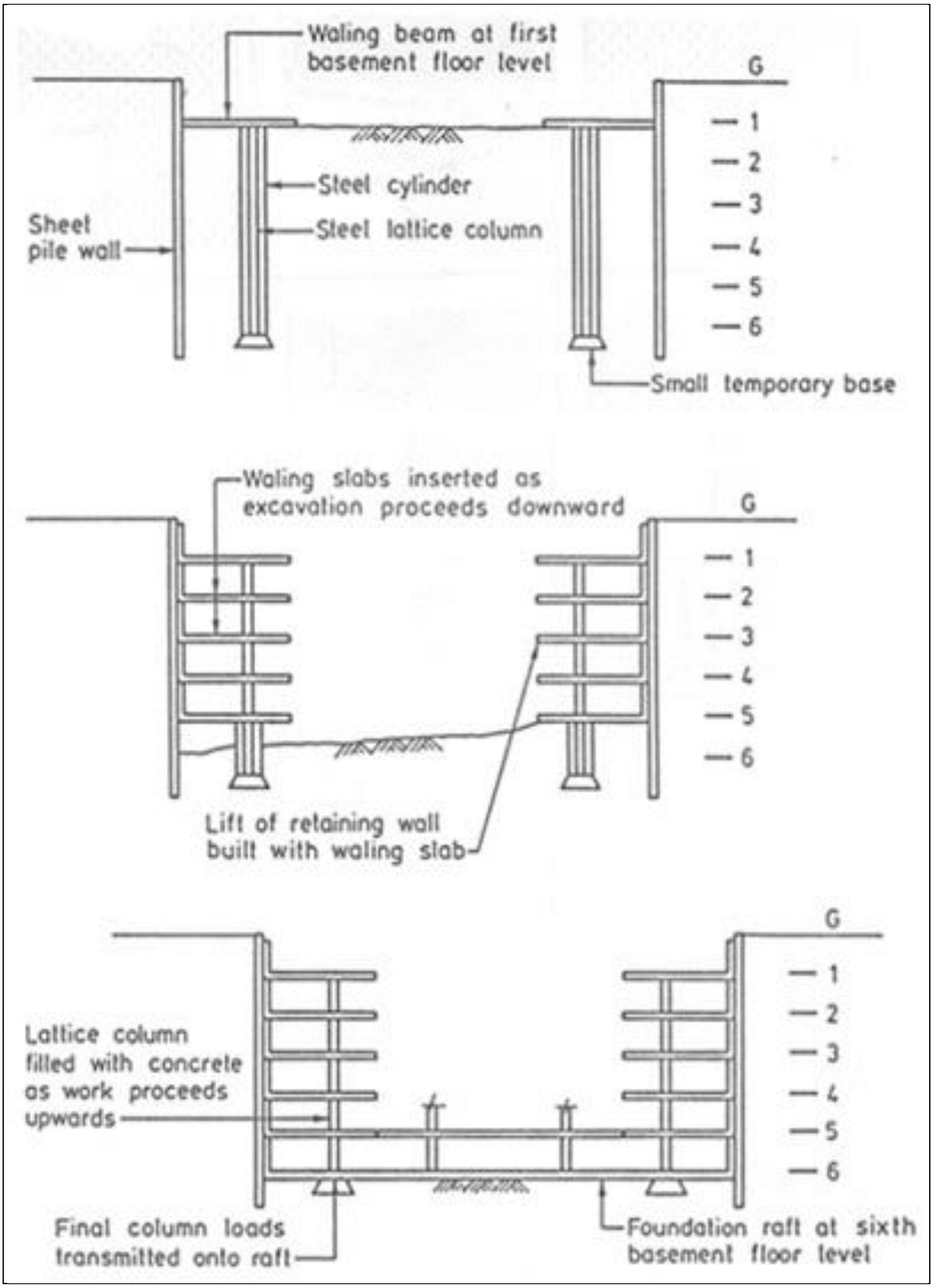

Figure 2.29: Typical processes for implementing top-bottom method- 6 basements [2]

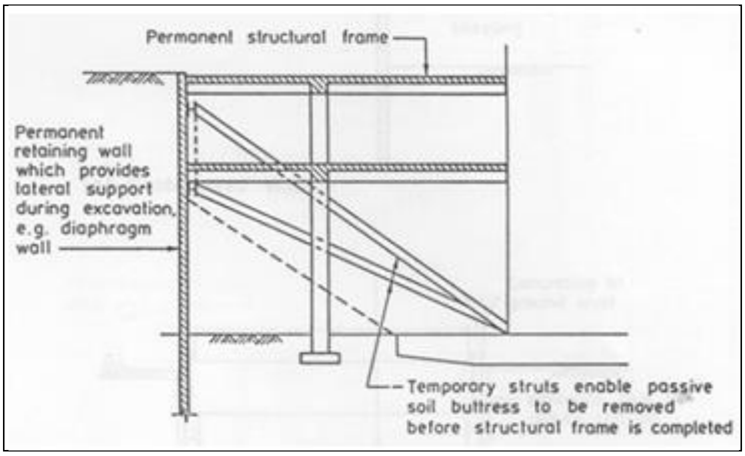

Figure 2.30: Simultaneously implementing of two level structural frames as a multi level secondary support of permanent retaining wall with temporary raker struts [2]

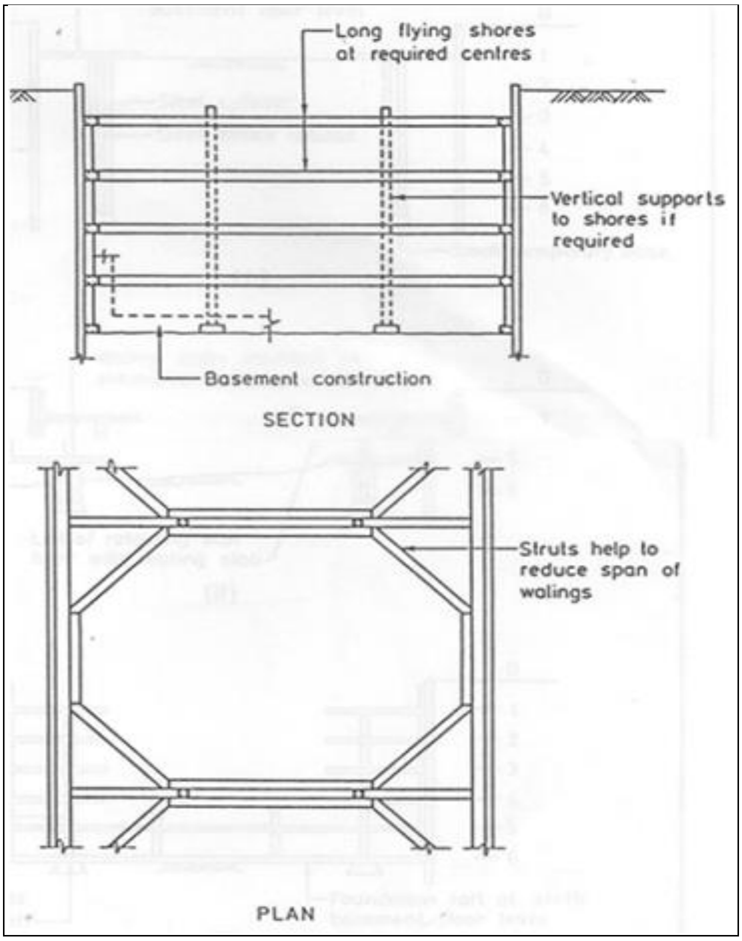

Figure 2.31: Schematically plan and section for implementing bottom-top method [2]

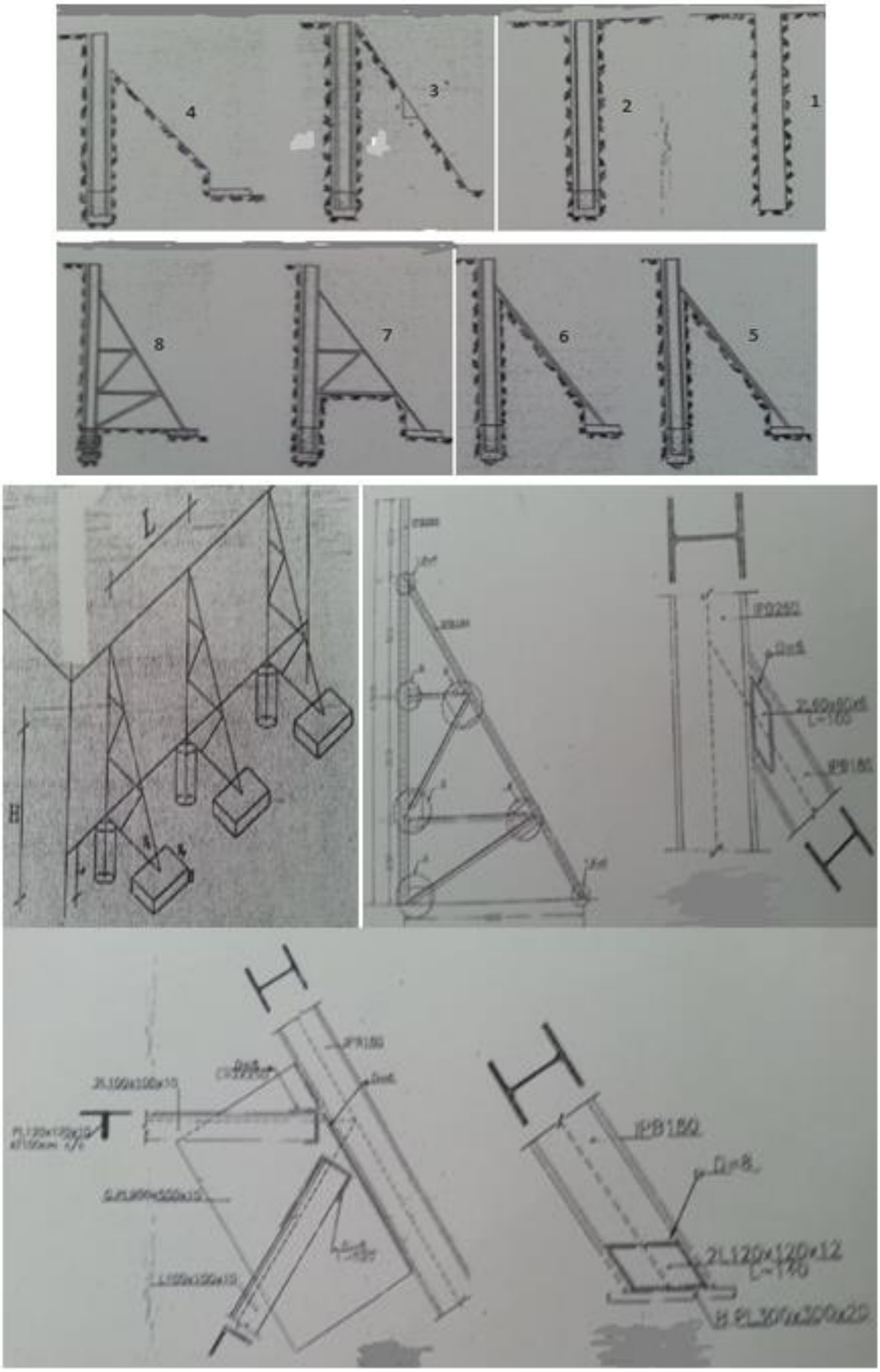

Figure 2.32: Typical processes and details to use soldier beam as a truss with lagging

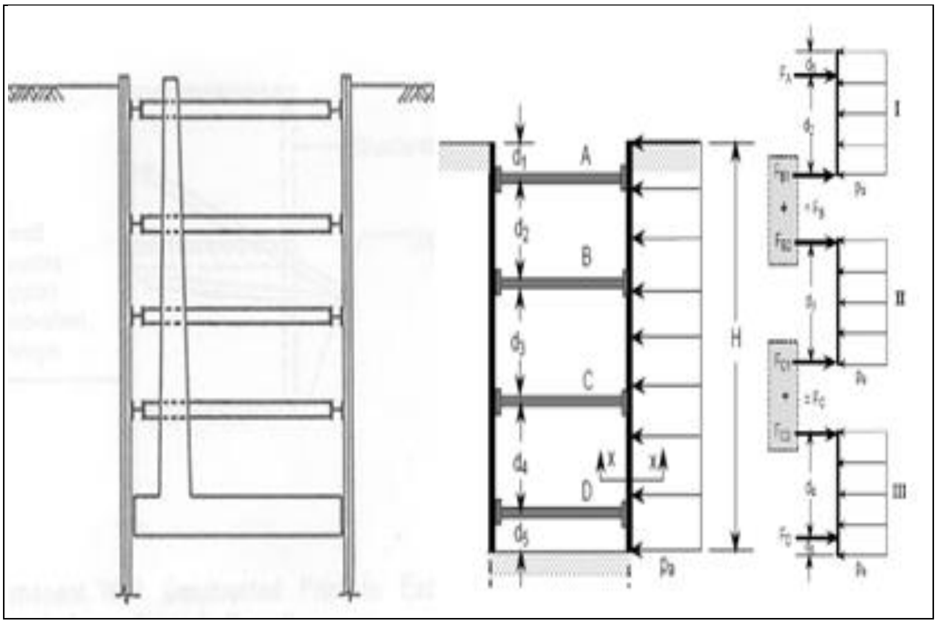

Figure 2.33: Use of a foundation alone to support the retaining wall in a narrow staged cutting and four level temporary bracing [2]

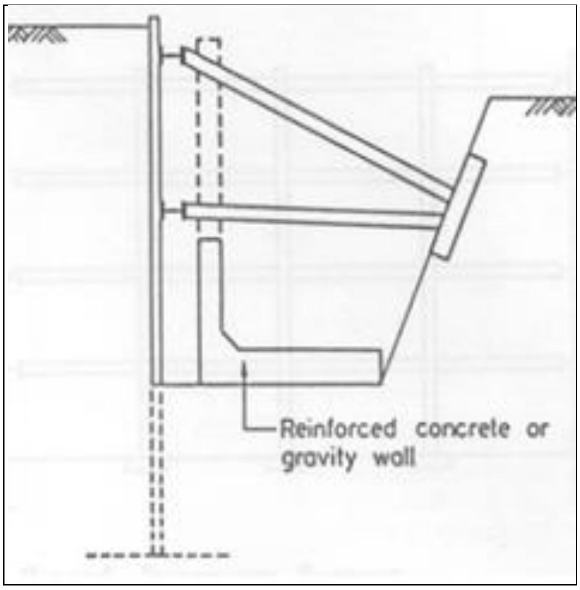

Figure 2.34: Use of a foundation to support the retaining wall in a narrow staged cutting and two level temporary bracing [2]

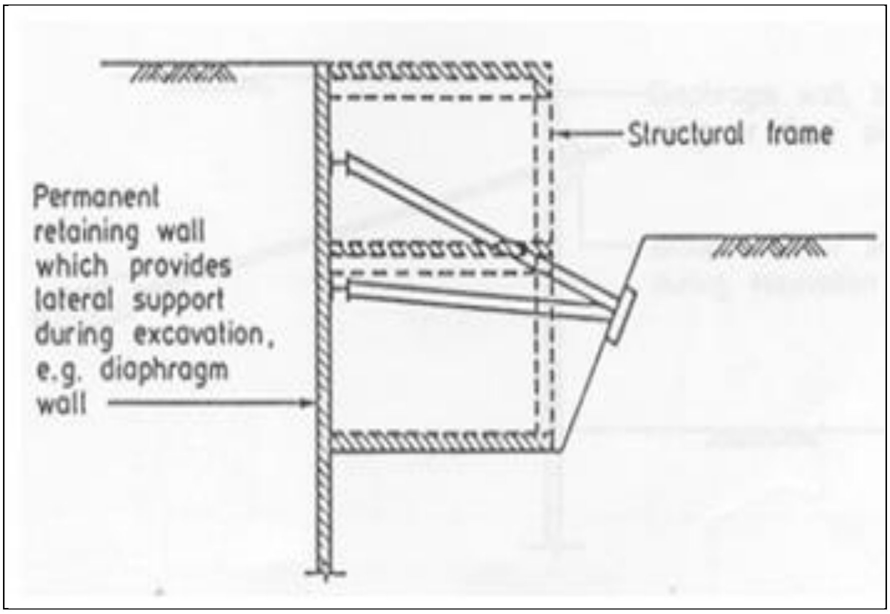

Figure 2.35: Permanent wall and two level temporary rakers for implementing main frame and slabs as a supporting system [2]

2.8 General geometry of deep excavation sites

In addition to the depth, it is considering that the volume and width of excavation could influence the selecting possible and economical methods and equipments. As a support for deep excavation there are a few categories. Cantilever retaining wall alone, strutted retaining wall, tied back retaining wall, Retaining wall with main frames' slabs are general division. Also combination of each kind of retaining wall with secondary and multi propped supports produce kind of sub-categories such as strutted tied back retaining wall. If there is enough space for require thickness of wall implementing, retaining wall alone is economic, otherwise if the width of excavation is narrow or lower than 12 meters, strutted retaining wall or tied back retaining wall or main frames' slabs depend on soil condition and situation could be economic. If the width of excavation is relatively large or more than 12 meters, and the depth of excavation is relatively small, the anchored retaining wall or raker braced retaining wall or main frames' slabs depend on soil condition and situation could be economic.

Tied back contiguous bored pile wall depend on soil condition could be the most economic between other cases of multi propped supporting systems if there isn't high level water table. Also other situations such as suitable availability into site or to materials or technology, and preventing from adjacent neighbors for anchoring could influence the initial geometrical based plan. A survey to summary of several deep excavations' general geometry case histories implemented by different methods could clarify the vision. The strutted diaphragm wall [34, 35] , by tie back diaphragm wall [32] , by strutted tie back wall [14] , by Contiguous bored pile wall [36] and by anchored contiguous bored pile wall is gathered from well-documented designs and constructed deep excavation cases in different projects that indicated in the next tables .

Summary of general geometry of deep excavation in two case histories under 12 meters (standard length of construction steel cross-sections) width implemented by Strutted diaphragm wall is indicated in table 2.15 [34]. However the need to columns and rafters may be probable. A review of general geometry of deep excavation in twelve case histories between 12 to 24 meters (two times of standard length of construction steel cross-sections) width implemented by Strutted diaphragm wall is shown in table 2.16 [34]. There is need to columns and rafters for possibility of strutting implementation in that case. As a Brief of general geometry of deep excavation in eight case histories more than 24 meters width implemented by Strutted diaphragm wall is indicated in table 2.17 [34, 35]. Summary of general geometry of deep excavation case history implemented by tied back diaphragm wall is shown in table 2.18 [32]. Brief of general geometry of deep excavation case history implemented by Strutted tie back diaphragm wall is indicated in table 2.19 [14]. Summary of general geometry of deep excavation two case histories implemented by contiguous bored pile wall is shown in table 2.20 [36, 37]. A brief general geometry of deep excavation case history implemented by tied back contiguous bored pile wall is shown in table 2.21. A review of general geometry of deep excavation for a metro station implemented by strutted (4 levels) diaphragm wall is indicated in table 2.22 [28].

Table 2.16: Summary of general geometry of deep excavation case histories under 12 meters width implemented by Strutted diaphragm wall [34]

N	**Case name (Strutted diaphragm wall)**	**Length B**(m)	**Width T**(m)	**Maximum total Depth of excavation**(m)	**Approximate total volume of excavation** (m^3)
1	Flagship Wharf	34	8	14	3800
2	Lurie [34]	64	7.4	11.8	5500

Table 2.17: Summary of general geometry of deep excavation case histories between 12 to 24 meters width implemented by Strutted diaphragm wall [34]

N	**Case name (Strutted diaphragm wall)**	**Length B**(m)	**Width T**(m)	**Maximum total Depth of excavation**(m)	**Approximate excavation volume** (m^3)
1	Song-san excavation	42	20	9.31	7800
2	East Taipei basin	68	23.5	14.1	22000
3	Taiwan Power Company	60	13.5	14.7	11000
4	Pudian Road	20.4	15.5	16.5	5000
5	Yanchang Road	18.1	15.5	15.2	4200
6	Syed Alwi	28	16	7.8	3400
7	MRT line in Singapore	20	20	16	6400
8	Farrer Park	21	22	17.5	8000
9	HDR-4 Subway	12.2	22	12.2	3200
10	Muni Metro Turnback	16	20.5	13.1	4200
11	Shanghai Bank building	43	19.3	15.2	12000

Table 2.18: Summary of deep excavation case histories geometry more than 24 meters width implemented by Strutted diaphragm wall [34, 35]

N	**Case name (Strutted diaphragm wall)**	**Length B**(m)	**Width T**(m)	**Maximum total Depth of excavation**(m)	**Approximate total volume of excavation** (m^3)
1	Formosa [26]	35	27	18.5	17000
2	Far East Enterprise[26]	70	24	20	33000
3	NTUH in Taiwan[26]	140	40	15.7	87000
4	Kotoku[26]	30	30	17	15000
5	Rochor Complex [26]	95	24	8.3	18000
6	Bugis MRT[26]	21	35	18	13000
7	Shaodao Temple[26]	21.5	26.5	18.5	10000
8	Metro station - China [27]	443.9	44.5	32.0	430000

Table 2.19: Summary of deep excavation case history geometry implemented by tied back diaphragm wall [32]

N	**Case name (tied back diaphragm wall)**	**Length B**(m)	**Width T**(m)	**Maximum total Depth of excavation**(m)	**Approximate total volume of excavation** (m^3)
1	Taipei County Administration center [32]	155	93	20	280000

Table 2.20: Summary of deep excavation case history geometry implemented by Strutted tie back diaphragm wall [14]

N	**Case name (Strutted tied back diaphragm wall)**	**Length B**(m)	**Width T**(m)	**Maximum Total Depth of excavation**(m)	**Approximate total volume of excavation** (m^3)
1	Naples underground station [14]	85.5	23.6	28.0 Maximum	54000

Table 2.21: Summary of deep excavation case histories geometry implemented by Contiguous bored pile wall [36, 37]

N	Case name (Contiguous bored pile wall)	Length B(m)	Width T(m)	Maximum Total Depth of excavation(m)	Approximate total volume of excavation (m^3)
1	Tan Tock Seng Hospital in Singapore [36]	200	140	15	420000
2	Siriraj Hospital on to Chao Phraya River bank [37]	225	130	10.85	310000

Table 2.22: Summary of deep excavation case history geometry implemented by tied back contiguous bored pile wall

N	Case name (tied back Contiguous bored pile wall)	Length B(m)	Width T(m)	Maximum Total Depth of excavation(m)	Approximate total volume of excavation (m^3)
1	Spring Mall (Northern Cyprus)	132	50	20	131000

Table 2.23: Summary of general geometry of a metro station deep excavation case history implemented by strutted diaphragm wall [28]

N	Case name	Excavation area (m^2)	Wall perimeter (m)	Maximum Total Depth of Excavation (m)	Approximate total volume of excavation (m^3)	Adjacent buildings minimum distance (m)
1	Hangzhou	12450	1016	32	398000	32

2.9 Monitoring instruments and equipments

Some of monitoring items and related instruments and equipments for deep excavation are indicated in table 2.23 [35].

Table 2.24: Monitoring items and related equipments [35]

Monitoring item	Instrument
Horizontal displacements of the retaining wall or adjacent soils	Inclinometer
Axial forces in struts	Transducer (for axial force)
Reinforcement stresses of the retaining wall	Stress gauge
Groundwater levels	Water-level tube, tape measure
Bottom heave	Settlement gauge, total station
Vertical displacements of adjacent buildings or top of the retaining wall	Level sensor, and theodolite
Settlements of soils	Level sensor, Theodolite or total station
Pore pressure	Pizometer, pressure meter

2.10 Index for deep excavation definition

Rolf Katzenbach et al. (2013) defined an index (T_{EI}) as a ratio of the resulting horizontal forces (H_{res}) and the oedometric modulus (E_S) which H_{res} includes the active earth pressure and the groundwater pressure. An excavation with T_{EI} index more than 0.4 is deep excavation [38].

$$T_{EI} = H_{res} / E_S > 0.4 \quad \text{(Eq 1.2)}$$

2.11 Conclusion

A simple definition of deep excavation has been presented. The truth of issues due to deep excavation and therefore necessity of supporting system is explained. Retaining wall as a first facade and struts or anchors depend on situation as a secondary facade which keep retaining wall as well as combination of the supports in multi-level jointly multi-staged excavation and their functions are described.

The necessary equipments for construction of kind of retaining wall and anchors as supporting system in deep excavation are explained briefly. Clamshell and grab for diaphragm wall, drilling machines for bored piles wall and anchorage, and hydraulic backhoe excavator for excavation are described so that gives collective data as a brief base to select appropriate machine for a situation and calculate the production rate of machine.

A relative classification of site geometry for deep excavation is proposed based on case histories which give relationship between geometry of site and deep excavation method.

The fundamental instruments for monitoring items in deep excavation are overviewed.

Chapter 3

RISK MANAGEMENT

3.1 Introduction

There is questions such as what is risk, what are risk categories, and which has relation to construction industries especially in deep excavation, how is systematic definition of damages in deep excavation, how can estimate expected damages in deep excavation.

This chapter contents:

1- Risk

2- Risk categorization

2.1- Business and financial risk

2.2- Project risk

2.3- Operational risk

2.4- Technological risk

2.5- Technical risk

2.6- External risk

2.7- Environmental risk

2.8- Organizational risk

2.9- Project management risk

2.10- Right of way risks

2.11- Construction risk

2.12- Strategic risk

3- Risk and corresponding damages

4- Expected damage at risk

5- Uncertainty and risk

6- Risk and decision-making

3.2 Risk

Risk is an uncertain happening into estimates of outcomes [39] or condition that can affect the outlook of project objectives [40] or likelihood of loss or damage [41] in a particular period of a process [42]. Risk could be separate as a situation of potential damage which can be accepted, refused, reduced its potential impact, and removed completely [43]. In a risky situation, the result of event is not known exactly but it is possible to determine the number, probability, and outcome value of possible results [44]. Risk is a considerate of the intensity of threat due to potential outcome that has already occurred, in a time of huge uncertainty, and quickly analyzes the situation [45, 46]. The ISO 31000 standards definition of risk is based on probability of occurrence and consequence of occurrence [46, 47]. If the probability of occurrence is not identified, the risk is undefined [46]. In negative case, the consequence of occurrence is how devastating, damaging, and unsuccessful, if it happens. There are different categories of risk.

3.3 Risk categorization

3.3.1 Business and financial risk

These are risks which affect business in its financial capability conditions. It includes risks related with the market which the business acts (market risk), in addition to the ability to finance intensification through loans (credit risk). These risks may be with a large number of financial tools [43, 45, 46, 47, 48].

3.3.2 Project risk

This risk affects project objectives by the client, planning, scheduling, construction, design, stakeholders, operational, and organizational factor. These risks affect project objectives such as cost, time, scope, and quality so that the expected objective could change significantly and even fail [45, 46, 47].

Project risks occurrence are firstly effect on one of the project objectives such as cost, time, scope and quality, or effect on a few objectives of project, and secondly effect on environment, and social that can be between neglected to high impact and costs directly or indirectly.

3.3.3 Operational risk

These risks include wide ranging of the probability of failures of operations from management failure, system failure, procedural failure, human error, to process inefficiencies. It may be on familiar terms as a part of overall risk management in project or business [45, 46].

3.3.4 Technological risk

This risk is associated with entrance of new technology to market. Entrance of new technology may cause some of competitors down out of market [45, 46].

3.3.5 Technical risk

This risk includes design imperfect, environmental analysis deficient or in error, unpredicted geotechnical issues, change requirements because of errors, inexact assumptions on technical issues in planning stage, study late or review in error, materials in error (resources, and logistics), geotechnical activities and report in error, foundation in error, structural designs imperfect or in error, unsafe waste site analysis partial or totally in error, require for design exceptions, consultant plan not up to section standards, situation susceptible solutions, fact sheet requirements (exceptions to standards) [43, 45, 46, 48].

3.3.6 External Risk

This risk includes adjacent owners' prevention against project implementing, conflicting cost of project, inconsistent scheduling of project, incoherent scope of project, not in agreement quality of project, local communities restraint, annually funding changes, change of political factors, lately change in stakeholders request, new work demand by new stakeholders, new stakeholders come forward, powerful stakeholders request additional requirements due to own purposes, threat of litigation, stakeholders desire cost over quality, and stakeholders come to a decision for time over quality [45, 46,47].

3.3.7 Environmental risk

This risk includes delay than expected time of agency action for permit, need to new information, altering in environmental regulations, changes in water regulation, changes in energy regulation, changes in material regulation, agency requires higher level review than unsaid, lack of specialized staff or worker, extraordinary site such as historical site, endangered variety, wetland present, disagreement on environmental grounds expected, specific quality at the plan level, water quality issue, expected negative impact by community, harmful waste, required preliminary site investigation, growth encouragement issues. Also this risk includes project in the specific zone or location such as coastal, attractive highway, near the wild prevented zone, near the charming river, in a floodway, in a flood plain. In addition, pressure to pack together the environmental timetable, and cumulative impact issues are belonging to this risk [45, 46, 47].

3.3.8 Organizational risk

This risk encompasses inexpert staff assigned, losing significant staff at critical point of the project, inadequate time to plan, unexpected project manager workload, internal delay to getting approvals, delay or error in decisions, functional units not accessible, filled to capacity, lack of considerate of multi-part internal funding procedures, not enough time to plan, priorities change on existing plan, new priority project inserted into plan, inconsistent cost, contradictory time, conflicting scope, and differing quality as a objectives [45, 46].

3.3.9 Project management risk

This risk comprises poorly definition of project target and need, incomplete project scope definition, defective project schedule, deficient cost estimate, ambiguity in definition or understanding of cost and deliverables, lack of control over staff priorities, too many projects for one manager, consultant delay, contractor delay, estimating error, scheduling error, unplanned work that must be accommodated, communication breakdown with project team, pressure to deliver project on an accelerated schedule, lack of coordination/communication, lack of higher management support, change in means staffing during the project, inexperienced workforce, insufficient staff, resource availability, local agency issues, public reaction and support, agreements [45, 46].

3.3.10 Right of way risk

These risks encompass utility relocation which may not happen in time [45].

3.3.11 Construction risk

This risk could include mistaken contract scheduling, permit work outlet, utility, surveys, and buried manmade objects/unidentified hazardous waste [45]. The construction industries risks could distribute to functional risks, structural risks, and contractual risks [48, 49].

3.3.12 Strategic risk

These risks include the macroeconomic, broke business decisions and direction [45].

3.4 Risk management

Risk management includes risk identification, register, assessment, and management that needs to a methodology which presents in chapter 4.

3.5 Risk and corresponding probable damages

Risk may be shown by the probability of a happening and its' corresponding damage that can be in term of money, lives (or casualties), or a unit based on work, energy, and/or material. Conventional risks of death could be in construction industries about 0.00018 (probability of risk occurrence is 18 death for 100000 worker), structural failure about 0.00000014, engineering occupations approximately 0.00011, earthquakes almost 0.000002, fire and burns accident around 0.00004, falls accident about 0.00009, impact of traffic to workshop almost 0.0003 , and whole population in all causes approximately 0.012 per annual [50]. Perfect engineering management especially in deep excavation indicates taking risks, but it must be suitable in identification, desirable precision in assessment, mitigation of the risks and averting failures or damages [39].

For a project, the location (underground, surface, territory, and country), size of project, material (resources, and logistics), and complexity (organization, technology, safety, hazard, etc) create potential risks. Also the speed of construction directly and the degree of familiarity with type of work (by manager, contractor, or workers) reversely are related to potential risk [48].

A research from Australia in 2005 indicated that the size of project has relatively reverse relation with risk rate due to occupational health and safety. The size of a project could be defined based on wage roll which its relation with risk rate is indicated in table 3.1 [51]. Accidental risk occurrence probability for projects with size less than $10 million wage roll is about 0.0398. The wage role may be among 15% to 75% for works cost in construction industries depend on conditions such as available technology and its price, abundance of labor, economic growth rate, level of material and energy prices, and government subsidy.

Table 3.1: Typical relation between project size and risk rate due to occupational health and safety and accident [51]

Project size (wage roll in $ million)	**Risk occurrence due to occupational health and safety accident**
Less than $10 million	3.98%
$10 million - $20 million	3.42%
$20 million - $30 million	1.31%
$30 million - $50 million	1.27%
Greater than $50 million	1%

3.6 Expected damage at risk

Let assume P_{DGFM} is probability of damage due to ground failure mode (DGFM) as a internal damage, P_{DGDE} is probability of damage due to ground failure effects on adjacent properties (DGDE) as a external damage, P_{DMTC} is probability of damage due to necessary machinery or tools collapses or unfavorable acts (DMTC) during working (it may includes overhead wages, rehabilitation expenses, debt installments, contractual delay penalty, depreciation, overhead residual, and the cost of lost opportunity) as a internal damage, and P_{DHSA} is probability of damage due to human safety accidents (DHSA). The mentioned probabilities of damages have relevant impact on project main objectives such as cost, time, scope and quality. In other word, the cost of probable DGFM (C_{DGFM}), the cost of probable DGDE (C_{DGDE}), the cost of required machines or tools collapses or unfavorable acts (C_{DMTC}), and the cost of human safety accidents (C_{DHSA}) imposed on project initial estimated direct costs (IEDC). Also the duration of work sleeping due to ground failure and returning to initial case (T_{iDGFM}), the duration of machine collapse (T_{iDMTC}), and the court probable temporary halt order (CTHO) imposed on project initial schedule and loss time. These are direct relevant influences on project that shall be realized on project payments accounting (except the cost of lost opportunity) even after project life cycle by probable court order. The mental pressure of heavy losses and casualties influences on scope of project and even business. The expected value of cost growing (EVCG) is proposed as:

$$EVCG = \Sigma\,(P_{iDGFM} \times C_{iDGFM}) + \Sigma\,(P_{iDGDE} \times C_{iDGDE}) + \Sigma\,(P_{iDMTC} \times C_{iDMTC}) + \Sigma\,(P_{iDHSA} \times C_{iDHSA}) \qquad \text{(Eq 2.1)}$$

where P_{iDGFM} is Probability of risk occurrence of ground or support failures, C_{iDGFM} is the cost of probable damages due to ground or support failures, P_{iDGDE} is probability of risk occurrence of ground failure effects on adjacent properties, C_{iDGDE} is the cost of probable damages due to ground or support failures effects on adjacent properties, P_{iDMTC} is probability of risk occurrence of necessary machinery or tools collapses or unfavorable acts during working, C_{iDMTC} is the cost of probable damages due to necessary machinery or tools collapses or unfavorable acts during working, P_{iDHSA} is probability of risk occurrence of human safety accidents, and C_{iDHSA} the cost of probable damages due to risk of human safety accidents.

Also the expected schedule increasing (EVSI) is proposed as:

$$EVSI = \Sigma (P_{iDGFM} \times T_{iDGFM}) + \Sigma (P_{iDGDE} \times T_{iDGDE}) + \Sigma (P_{iDMTC} \times T_{iDMTC}) + +\Sigma (P_{iDHSA} \times T_{iDHSA}) \quad \text{(Eq 2.2)}$$

where P_{iDGFM} is Probability of risk occurrence of ground or support failures, T_{iDGFM} is the duration of risk of ground or support failures, P_{iDGDE} is probability of risk occurrence of ground failure effects on adjacent properties, T_{iDGDE} is the duration of risk of ground or support failures effects on adjacent properties, P_{iDMTC} is probability of risk occurrence of necessary machinery or tools collapses or unfavorable acts during working, T_{iDMTC} is the duration of risk of necessary machinery or tools collapses or unfavorable acts during working, P_{iDHSA} is probability of risk occurrence of human safety accidents, and T_{iDHSA} is the duration of risk of human safety accidents.

3.7 Uncertainty and risk

Uncertainty is a situation in which the purpose of the number of possible results, the probability of occurrence, and the value of consequence of probable result, is not possible while in a situation of risk, the result of a happening is not known accurately but it is probable to determine [52].

3.8 Risk and decision-making

The choosing of a course of action from among alternatives (acts) is decision-making [44] which may have uncontrollable future events [53]. There are techniques to improve the quality of decision-making under uncertainty which are risk analysis, decision tree, and preference theory (assumption instead of purely statistical probabilities) [44]. Decision-making and risks are jointly and the level of risk depends on prior information that could be retrieved in studying stages. Risk is often studied by expert or a team of experts based on stages from fact to value and proposed to decision maker. Fact starts from evidence, converts to knowledge base, then risk assessment in relatively wide spectrum, and finally reaches for decision maker appraisal [54]. A model for linking the various stages in the risk informed decision-making is shown in figure 3.1 [54]. The importance of a decision might be evaluated by its impact which may be responsibility, monetary damage, fatalities, goals, job security, deputies, losses in different levels, more income, and so on [44].

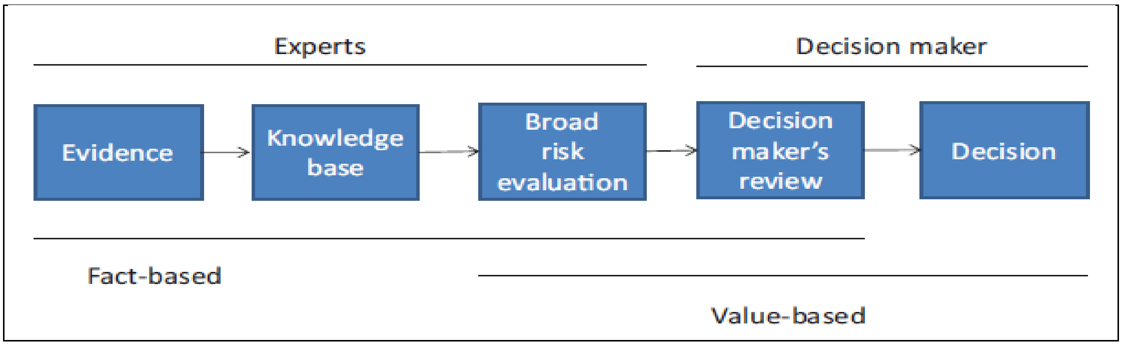

Figure 3.1: A model for linking the various stages in the risk informed decision-making [54]

3.9 Conclusion

Risk definition and categorization is overviewed which is a straight guide to risk origin and scope. Also the difference between risk and uncertainty is overviewed which in uncertainty there is not any way to estimate risk occurrence probability.

The expected value of cost growing (EVCG) as an expected damage due to deep excavation risk sources as divided into internal, external, and accidents is proposed. Also the expected schedule increasing (EVSI) is proposed.

The relation between project size and risk rate due to occupational health and safety and accident is overviewed.

Chapter 4

METHODOLOGY TO STUDY RISK MANAGEMENT IN DEEP EXCAVATION

4.1 Introduction

There are questions such as what methodology could be defined for study of risk management in deep excavation, how it can be risk identification in deep excavation, how can risk register in deep excavation, how can risk assessment in simple deterministic method, and which response is more suitable.

Risk identification in term of geotechnical or productive identification, risk register in term of geotechnical or productive failures, damage division in deep excavation (in term of internal, external, and accident), expected damage, and risk assessment based on new formula based on factor of safety (FOS) are proposed in this chapter briefly. This chapter contents:

1- Methodology to study risk management in deep excavation

2- Grasp increase

3- Identifying potential risks (risk identification)

4- Risk register

5- Analyzing the situation (risk assessment)

6- Risk response plan

4.2 Methodology

The methodologies to study risk management depends on situation and data existing [55]. A methodology of study is defining objectives to decrease the risk probability and impacts of the failure modes or failure modes effect as negative effects on project objectives with steps are:

1- Grasp increase by gathering existence science, learning remedies, and deduce relevant to reducing uncertainty. Grasp increase by study well-documented recent case histories, lessons learned, and deduce relevant to reducing uncertainty. Grasp increase by study of early mentioned statements and use for the purpose of preparing roles and responsibilities in contracts for future projects to decrease uncertainty.

2- Identifying potential risks by interviewing people with similar project experience, or by surveying the case studies for risk assessment and analysis purpose.

3- Risk register by output of risk identification as a document include the results of various risk management process in a table with related information about events, category, root cause, triggers, risk owner, probability and impact of each risk occurring, and the standing of each risk.

4- Analyzing the situation of each project with effective risky factors to estimate the range and minimum cost and scheduling due to risks that analysis could be into qualitative or quantitative.

5- Preparing risk response plan

6- Monitoring and controlling risks throughout the project.

4.3 Grasp increase

Grasp increase could be obtained by gathering and studying existence science about deep excavation in geotechnical engineering such as soil identification and classification, ground layers settlement , shear strength, unsaturated and saturated soils, lateral earth pressure, actions and effect of actions, combination of actions and corresponding effects, limit state design of supporting system (diaphragm reinforced concrete wall, bored pile wall, anchors, and struts), stability of supporting system and ground, and well-documented case histories about failures and their effects. Also existence science and information about excavation methods and equipments such as clamshell (hydraulic and/or cable), vertical rotary drill rig, horizontal directional drills, hydraulic backhoes, and vibro-hammer should be prepared. Material requirements such as concreting, steel reinforcement bars, high tensile strength steel bars or strands, steel beams or sections profiles, bentonite, water and also their required tools should be prepared.

4.4 Identifying potential risks

Risk identification for deep excavation relies mostly on past experiences that were well-documented. Excavation with the potential to cause an unreliable or an unsafe effect could be a hazard. There are often some questions for stakeholders (e.g. insurances, contractors, and construction managers) such as: how serious deep excavation problems are, how can comparison different deep excavations hazard, how registering priorities to decide appropriate level of deep excavation safety management, and how demonstrate monitoring and control measures have the maximum influences. Source of risk is tracking by subsurface condition, excavation depth, supporting system type, construction technology, method of excavation, and interaction between excavation stages and supporting response.

4.5 Risk register

Deep excavation has the potential to cause an unreliable or an unsafe effect (failure, crack or settlement in adjacent buildings, street, utilities) that could be Sliding, overturning, bearing capacity, Basel heave, bottom heaven, artesian pressure heaven, hydraulic heave, boiling, upheaval, Cracking, deformation, and kind of interior structural failures or effects. Excavation activates actions with different situations which are persistent (permanent and variable such as dead and live loads), transient (temporary condition such as execution or repair period), accidental (exceptional condition and usually short duration such as impact, fire, explosion), and seismic. Actions have favorable or unfavorable effects and affected by soil parameters on supporting system of deep excavation. Actions could include weight of soil and rock, removal of load of excavation of ground, earth pressure, dead and live loads, stresses in ground, ground water pressure, free water pressure, combined forces, and indirect actions as a lateral earth pressure and/or uplift. Imagination of lateral earth pressure are based on category of active (retaining wall moving away from the soil it retains), passive (retaining wall moving toward the soil it retains), and/or at-rest (the wall is not moving away or toward the soil it retains) conditions. Basically, at-rest pressures exist when the top of the wall is fixed from movement. Active and passive pressures are assumed relatively when the top of the wall moves at least 0.001 of height of wall in the direction away from, and toward the soil it retains, respectively. Lateral earth pressures (stress due to actions) area for soil or pore water component is triangle with the base of the triangle at the base of the wall (figure 4.1-a). Pressure areas for earthquake pressures are an upside-down triangle (figure 4.1-b) and surcharges have rectangular shape area (figure 4.1-c). Furthermore, production rate, duration, and unit cost are other unreliable factors in deep excavation.

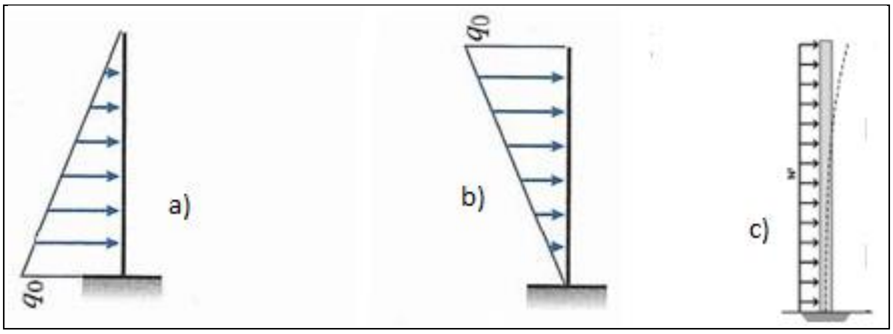

Figure 4.1: lateral pressures areas: a) soil or pore water
b) earthquake c) surcharges

4.6 Analyzing the situation

The principle factors to option of risk analysis method depend on the type and size of the project, the information obtainable, the cost of the analysis and the time existing to take it, the experience of the analysts, and purpose of analysis [56].

By identifying a project variable and carry out a sensitivity analysis, it gives variable limits and its' likely to vary [56]. Risk sources are defined as variables such as ground condition, or effect of ground failure (adjacent building collapse by excesses deformation of retained soil) that have a potential to cause damage. Failure modes and effects sensitivity analysis is technique for identifying risks and risk management of possible errors causes in a construction project, or construction process such as deep excavation in order to systematically improvement of processes and reduces project costs and duration [57]. Risk reduction may be possible by obtaining additional information, performance additional tests/simulations, and allocating additional resources [56]. Sensitivity analysis is used for analyzing with determining range of possible amounts for identified variables as defined by scientific approach.

Failure of ground or support as a risk should be demonstrated by use of calculation (analytical, semi-empirical, and numerical), measures, tests of experimental models, observational methods, and combination of those activities. There are recommendations to be checked in deep excavation [58, 59, 60, 61, 62, 63]:

1- Loss of ground or support static equilibrium as a rigid body in which the shear strength of soil or rock is insignificant in providing resistance (EQU)

2- Internal failure or excessive deformation of the structure of support (STR)

3- Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO)

4- Loss of equilibrium of the structure or ground due to uplift by water pressure (UPL)

5- Hydraulic heave in the ground due to hydraulic gradients (HYD)

Limit states UPL and HYD are only related to situations involving groundwater and hence only apply to geotechnical designs. Limit states should be based on ultimate limit states (ULS), serviceability limit states (SLS), durability limit states (DLS), and overall stability (OS). ULS includes Rupture or Excessive Deformation of Structure or Ground (REDSG). Eurocode 7 (EC7) identified three design approaches based on REDSG [51].

Geotechnical failure (risk) occurrence during deep excavation could be function of excavating depth, weak supporting system, exceed actions (case of load, magnitude, direction, changing, intensity, velocity of applying, loads combination, etc), exceed displacements, groundwater level fluctuation (dewatering, evaporation due solar radiation, and soil suction) , materials (origin, kind), climate changing (short term to long term), and position in soil.

4.6.1 The common probable causes of risk

The common probable causes of risk failure or deformation in retained soil as a risk could be one or more than one of the following factors during deep excavation: incorrect design (unsuitable plan and details selection, poor underground investigating layout, powerless modeling and insufficient parameters, and robustness), material defects (poor condition of reused steel struts, unsuitable concrete, under designed specified anchor, e.g.), construction errors (error in excavation depth definition, error in construction planning, error in implementing sequence of works, un-investigated struts connections, uncontrolled tolerances, inaccuracy to check adjacent building condition such as possibility of crack, settlement and related problems), poor maintenance, natural disaster (earthquake, wind, rainfall, and flood, e.g.), time (short term to long term environmental effects, historical state), exceed actions (case of load, magnitude, direction, changing, intensity, velocity of applying, loads combination, etc), and sabotage (site security problem, e.g.).

4.6.2 Geotechnical risk occurrence probability

Due to influence of probability on determining of actions and resistant, factor of safety (FOS) may be assumed random variable. Safety factor equal one is limit stable condition to be sure occurrence of failure in deterministic method.

FOS = Resistant / Action = R/A (Eq 4.1)

The probability that the system of deep excavation is not failing is then:

P (FOS >1) = P(R > A) (Eq 4.2)

Let us assume: FOS_{min} as minimum factor of safety among different factor of safeties, and FOS_{sure} as recommended sure factor of safety according to experiences:

If $FOS_{min} \geq FOS_{sure}$ then risk occurrence probability P = 0,

If $FOS_{min} < FOS_{sure}$ and $FOS_{min} > 1$ then risk occurrence probability $0 < P < 1$,

If $FOS_{min} < FOS_{sure}$ and $FOS_{min} \leq 1$ then risk occurrence probability P =1.

Based on probability definition for situations into $FOS_{min} < FOS_{sure}$ and $FOS_{min} > 1$, the geotechnical risk occurrence probability is proposed as:

$P_{Risk\ occurrence} = (FOS_{sure} - FOS_{min}) / (FOS_{sure} - 1) \leq 1.0$ (Eq 4.3) where FOS_{sure} -1 is numeric range of sample space and FOS_{sure} - FOS_{min} is numeric range of event.

4.6.3 Risk consequence

Failures as risk have damages which are risk consequence. Risk consequence is seen on affected adjacent building value, adjacent utilities value, occupation safety rules regard degree, and internal properties.

4.7 Risk response plan creation

Risk management requires an approval that uncertainty exists. Risk management creates a structure response to risk in terms of alternative plans, solutions, and contingencies [64]. Risk management is a judgment process requiring mind and creativity. Risk management generates a practical (and sometimes diverse) approach in project employees by preparing them for risk events, instead being taken by astonishing when they happen. Risk management encourages provision of appropriate contingencies and consideration of how they should be managed [56]. After appraisal of risk management response plan, systematically monitoring and control of risk, integration if it is require to risk reduction, and well-documenting are necessary for achieving deep excavation. Risk response plan creation is dealing with risk really and needs to define strategies in order to decision-making such as [64]:

1- Avoiding the risk by not to start, or not to continue deciding with the risky activity or change the plan, or condition in order to protect the project objectives such as change resource or time, adopt a familiar subcontractor, clarify requirements, reduce scope.

2- Accepting the risk and consider it in costs in order to create an opportunity.

3- Mitigating the risk by reducing the probability and impact of risk before it take place, implementing new course of action, choosing more stable supplier, change condition, try to eliminating the risk source with relatively low cost, converting or shift the likelihood, and/or the consequences.

4- Transferring the risk by sharing the risk with another party or parties including contractors (liability) and risk buyers (insurance, bonds, guarantee, and warrantee). There are categories for the sharing of risks in contracts which are legal (changed condition clauses be integrated in contract), financial, technical, measurement, and contract administration [65].

4.8 Flow chart to describe proposed methodology

A flow chart for briefly clarifying the processes of the proposed methodology is prepares which is illustrated in figure 4.2.

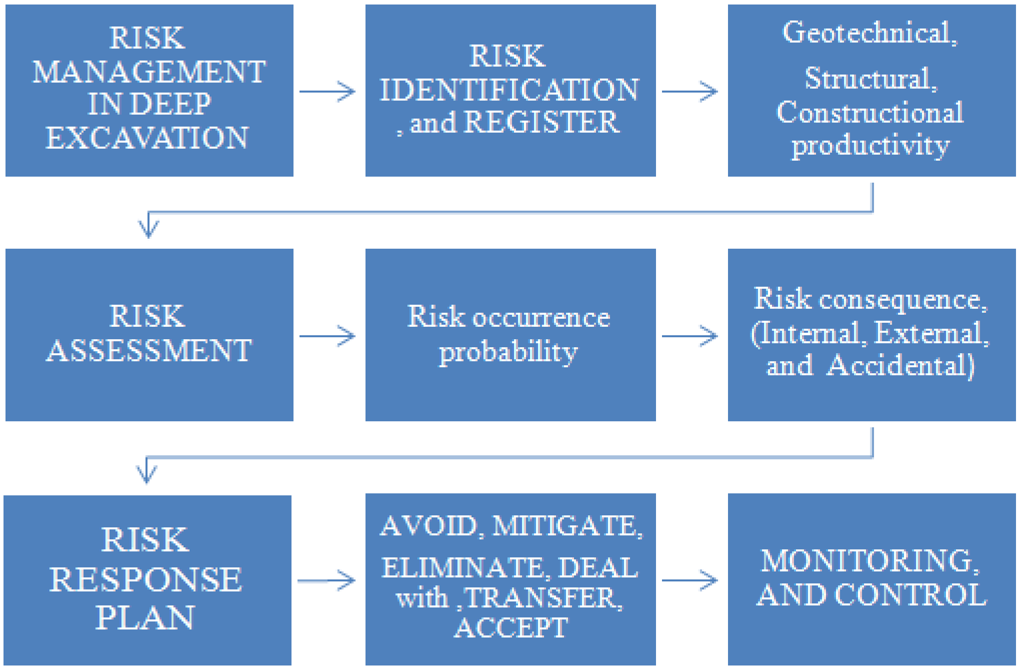

Figure 4.2: Flow chart for briefly describing the processes of the proposed methodology for risk management in deep excavation

4.9 Conclusion

A Methodology to study risk management in deep excavation is proposed which has explained in five steps.

A method based on probable safety factor is proposed for estimating risk occurrence probability in order to use in sensitivity analysis with determining range of possible amounts (low and high) for identified variables and estimating expected consequence.

Risk response plan is overviewed and explained generally for using in deep excavation which is a strategic procedure jointly monitoring and controlling.

The fundamental steps for geotechnical risk is proposed: site investigation and identification as risk identification, study of failure modes as risk register, failure effects, and related kind of damages, then risk analysis and calculating expected damages, next preparing risk management response plan (acceptance, mitigation, transfer, and avoidance strategies) in order to propose for decision-maker.

Chapter 5

SUBSURFACE IDENTIFICATION, CLASSIFICATION, AND SITE INVESTIGATION

5.1 Introduction

Underground soil may be source of risk for deep excavation which with different kinds of soils has different behavior and response (see chapters 6, 7, and 8). Obviously risk identification in multiplicity of source needs to identification and classification. Although there are classification systems in geotechnical engineering such as unified soil classification system (USCS) or other systems which can be used in risk studies as a base, but risk studies need to have a confidence interval or low and high quantities in risk identification in addition. Furthermore there are significant differences in observations of one defined parameter such as elastic modulus, shear strength, or unit weight of different samples from a specific site which building codes recommend more number of tests to determine that parameter for engineering design but in construction there are situations or change of situation which need to forecast and decision-making for future strategy. For that reasons it is need to identify range of quantified parameters for risky situation analysis. This chapter content practical glance to:

1- Subsurface identification and classification (Clays, granular, and intermediate soils)

2- Site geotechnical investigation

5.2 Subsurface identification and classification

Identification and classification of soils could be function of factors which are known relatively, and used in practice. Factors such as particle appearance and size, general identification (Unified system of soil classification), relative density of non-plastic soils (indexes with corresponding properties), and clays as a main factor in underground behavior (including expansion index, liquid limit, plasticity index, swelling potential, specific surface area, thickness, compressive strength, cohesion , steadiness index, safe vertical height after excavation, and corresponding relationships) could be used in an effective identification and classification.

The first step of soil identification and classification is size and appearance that they are boulder, cobble, gravel, sand, silt, to clay, and organic soils classes but it is not sufficient. Table 5.1 presents the soil identification and classification based on particle size, appearance, and main names of Unified soil classification system (USCS) [20]. Classes could be determined by direct measuring for rock, by sieve test diameter measuring for gravel, sand, silt and clay, or hydrometric test for very small particles of clay. The percentile of each class in a sample could be determined by initially separating rocks, then sieve test and weighting retained on each size and calculating percent passing. Unified soil classification system (ASTM D2487-06) identified soils in divisions. Firstly it classified soils based on particle size as coarse-grained soils (less than 50% pass No.200 sieve- 0.75mm) and fine-grained soils (50% or more pass No.200 sieve- 0.75mm). Secondly it classified coarse-grained soils based on percentage coarse fraction less than 6mm (¼ inch) and percent smaller than 0.75mm (No.200 sieve) and also it classified fine-grained soils based on liquid limit values (dry strength and shaking in practice) [66, 67] .

Table 5.1: Soil classification based on particle appearance and size [19, 80]

Name	**Size or appearance category**	**Comments**	**USCS**[37]
Boulder	Diameter > 305 mm (12 inch)	Rock particle	
Cobble	Diameter > 76 mm(3 inch) , Diameter ≤ 305mm(12 inch)	Rock particle	
Gravel (G)	Diameter > 6mm(1/4 inch) , Diameter ≤ 76mm(3 inch)	Coarse-grained soil	GW,GP,GM, GC, and their mix (1)
Sand (S)	Diameter > 0.75mm(sieve no:200), Diameter ≤ 6mm(1/4 inch)	Coarse-grained soil	SW, SP, SM, SC, and their mix (2)
Silt (M)	Diameter > 0.002mm, Diameter ≤ 0.75mm(sieve No.200)	Fine-grained soil	ML, MH
Clay (C)	Diameter ≤ 0.002mm	Fine-grained soil	CL,CH, CL-ML
Organic soil (O)	Vegetable matter, fibrous texture, normally dark color, spongy feel	Fine-grained soil	OL, OH, Pt
GW-GM, GW-GC, GP-GM, GP-GM, GP-GC, GC-GM, SW-SM, SW-SC, SP-SM, SP-SC, SC-SM			

Clays are main factors in underground behavior. Volume of clay after shrinkage limit (conversion of solid to semi-solid state) desires to expansion. Factors such as expansibility index, liquid limit, plasticity index, swelling potential, specific surface area, thickness, compressive strength, cohesion, steadiness index (very soft to very stiff and hard), safe vertical height after excavation, could be used to identification and classification of clays in deep excavation. Liquid limit and plasticity Index represent the ability of clay to absorb water. Steadiness of clay from very soft to hard could be compared with compressive strength, cohesion, theoretical safe vertical height, and the occupancy safety (OHSA) soil type. The occupancy safety searches risks of working and manages with regulated preventing work nasty accidents and deals with providing the protection of people during working. OHSA classified excavation soils to A, B, and C types in order to identify harmless vertical cut heights in very soft to hard state (steadiness) of clays. Considering the importance of clays in soil behavior, a simple collection of data ranges from kind of geotechnical reports, books, and essays could be gathered in term of dry unit weight, cohesion, compressive strength, safe height, SPT test results, and OHSA soil type that is shown in table 5.2 [4,7, 8, 10, 14, 28, 33, 37, 66, 67, 68, 69,70]. Clays in a classification based on expansive properties are indicated in table 5.3 [66, 67]. Characteristic identification and classification of three main groups of clays are indicated in table 5.4 [66, 67]. Figure 5.1 shows the fine-grained soil plasticity chart (ASTM D 2487-06) that determine the situation of fine-grained soils for plastic characteristic based on liquid limit vs. plasticity index.

Table 5.2:The range of data for clays [4,7, 8, 10, 14, 28, 33, 37, 66, 67, 68, 69,70]

Clay Soils	**Dry unit weight** γ_{dry} (kN/m^3)	c_u **cohesion** (kN/m^2)	**Safe height** (m)	**Compressive strength** (q_u) (kN/m2)	**SPT-** N_{30}	**OHSA type**
Hard clay	19.6 – 21.6	> 192	> 24	>383	> 32	A
Very stiff clay	17.6 – 20.2	96 – 192	12 - 24	192 – 383	16 - 32	A
Stiff clay	16.9 – 19	48 - 96	6 - 12	96 – 192	8 - 16	B , A
Medium clay	16.2 – 18.4	24 - 48	3 - 6	48 – 96	4 - 8	B
Soft clay	15.6 – 17.8	12 - 24	2 - 3	24 – 48	2 - 4	C
Very soft clay	14.8 – 17.2	0 - 12	1.5	< 24	0 - 2	C

Table 5.3: Clays in a classification based on expansive properties [66, 67]

Clay Expansive Classification	**Liquid limit** (%) (LL)	**Plasticity index** (%) (PI)	**Swelling potential** (%) (SWP)
Low	Less than 50	Less than 25	Less than 0.5
Moderate	50 - 60	25 - 35	0.5 – 1.5
High	More than 60	More than 35	More than 1.5

Table 5.4: Characteristic identification and classification of three main groups of clays [66, 67]

Clay type	**LL** (%)	**PI** (%)	**Specific surface area** (m^2/gram)	**Thickness** (A^o)	**Lat.Dim** (A^o)
Kaolinites	33-65	8-30	10 - 45	10^2 - 10^3	1000 - 20000
Illites	30-120	10- 45	38 - 100	50-100	1000-5000
Montmorilonites	25-710	14-631	59 - 800	10-50	1000-5000

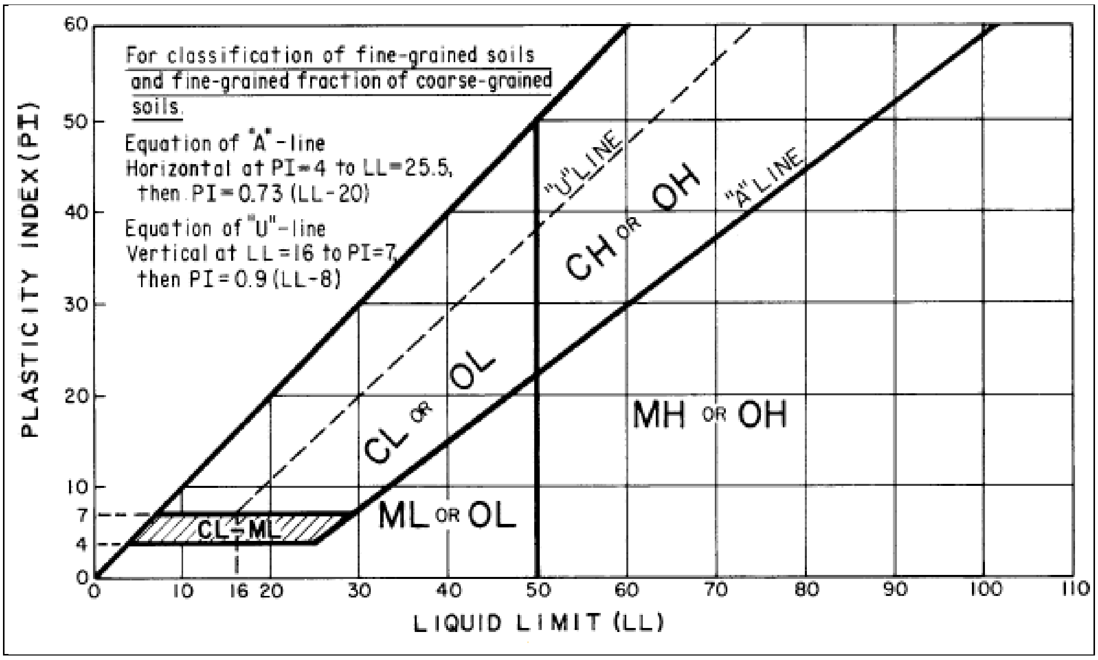

Figure 5.1: Fine-grained soil plasticity chart based on USCS (ASTM D 2487-06)

For granular soils another range of data collection in terms of dry unit weight, internal friction angle, SPT, drained module, and relative density is gathered and indicated in table 5.5 [10, 28, 4, 14, 37, 66, 67, 7, 69, 33, 70, 71]. Table 5.6 indicates the range of data for intermediate soils (Sand+Silt+Clay) which percentage of clay is more than 50% [72]. The range of data for silty sand, silty clay, sandy clay, clay with silt, silty clay with sand, and saturated normal clays is gathered in table 5.7 [4, 7, 9, 10, 14, 69, 73, 74, 75]. The range of lateral earth pressure coefficient by rankine's method as active (K_a) and passive (K_p) considering horizontal terrace (similar to urban ordinary condition) from formulas or accepted specified recommendations for clays by ground water level which is proposed by Katsura et al (1995) [17], external friction angle (δ) between concrete wall ($\phi'\times 3/4$) and soils or between steel sheet pile wall and soils, are collected and shown in tables 5.8. The unit of external friction angle (δ) and Internal friction angle (ϕ) is degree.

Table 5.5: The range of data for granular soils [10, 28, 4, 14, 37, 66, 67, 7, 69, 33, 70]

Soil	**Dry unit weight** (kN/m^3)	**Internal friction angle**[a] ϕ (deg)	**SPT-N_{30}**	**Drained moduli**[b] **(E)** (MPa)	**Relative density**[c] (%)	**Moist. density**[d] (gr/cc)
Very loose sand	11.0 – 14.2	30 - 34	0 - 4	< 10	0 – 15	1.12-1.6
Loose sand	14.0 – 15.6	33 - 37	4 - 10	10 - 20	15 – 35	1.44-1.8
Medium sand	15.4 – 20	36 - 39	10- 30	20 - 30	36 – 65	1.76-2.1
Dense sand	16.9 – 20	38 - 42	30 - 50	30 - 60	66 – 85	1.76-2.3
Very dense sand	18.3 – 23.5	41 - 44	Over 50	60 - 90	85 – 100	2.1-2.4

a) Values (degree) given are valid for sands. For silt soil a reduction of 3°should be made. For gravels 2° should be added [18].

b) E is an approximation to the stress and time dependent secant modulus. Values given for the drained modulus correspond to settlements for 10 years [18].

c) Relative density = $\frac{emax - e}{emax - emin} \times 100$ where e is void ratio

d) Approximate moist density

Table 5.6: The range of data for intermediate soils (Sand+Silt+Clay, Clay percent more than 50%) [72]

Intermediate soils [71]	**N_{30}** (SPT)	**c_u**(kN/m^2)	**ϕ** (degree)
Loose (Sand + Silt+ Clay)	< 10	12	8
Medium (Sand + Silt+ Clay)	10 -30	5 - 48	8-12
Dense (Sand + Silt + Clay)	>30	48	12

Table 5.7: The range of data for silty sand, silty clay, silty clay with sand, sandy clay, clay with silt, and saturated normal clays [4, 7, 9, 10, 14, 69, 73, 74, 75].

Soil	**Dry unit weight** (kN/m^3)	**Internal friction angle ϕ** (deg)	**c_u** (kN/m^2)
Loose silty sand	12 - 14	27 – 30	
Dense silty sand	16 – 21	30 - 34	
Sandy clay	18 – 21.7	34	
Silty clay	19.1 – 21.9	15	12
Medium clay with silt	16.2 – 16.7	8 - 12	22 - 28
Soft silty clay	14 – 18.2	3 - 27	3 - 22
Medium silty clay with sand	15.5 – 18.7	13 - 22	13 - 25
Stiff silty clay with sand	16 – 19	18 - 30	11 - 43
Saturated normal clays	17.6 – 18.6		20 - 80

Table 5.8: The range of K_a , K_p (terrace is horizontal), and external friction angle (δ) between concrete/soil, and between steel/soil for kinds of soils

Soil	**ϕ**	**K_a**	**K_p**	**δ concrete**	**δ steel**
Very loose gravel	32 - 36	0.26 - 0.30	3.86 - 3.33	22.5 – 25.5	20
Loose gravel	35 - 39	0.23 - 0.27	4.39 - 3.69	24.7 – 27.7	20
Medium gravel	38 - 41	0.21 - 0.24	4.83 - 4.20	27 – 29.25	20
Dense gravel	40 - 44	0.18 - 0.22	5.55 - 4.60	28.5 – 31.5	20
Very dense gravel	43 – 46	0.16 - 0.19	6.13 - 5.29	30.75 - 33	20
Dense sandy gravel	50	0.13	7.59	36	20
Very loose sand	30 - 34	0.28 - 0.58	3.54 - 1.73	21 - 24	20
Loose sand	33 - 37	0.25 - 0.30	4.02 -3.39	23.3 – 26.25	20
Medium sand	36 - 39	0.23 - 0.26	4.39- 3.85	25.5 – 27.75	20
Dense sand	38 - 42	0.20 - 0.24	5.04- 4.20	27 - 30	20
Very dense sand	41 – 44	0.18 – 0.21	5.55- 4.81	29.3 – 31.5	20
Loose silty sand	27 - 30	0.33 - 0.38	3.0 – 2.66	18.75 - 21	20
Dense silty sand	30 - 34	0.28- 0.33	3.54 – 3.0	21 - 24	20
Very loose silt	27 - 31	0.32 – 0.38	3.12- 2.66	18.7 – 21.75	20
Loose silt	30 - 34	0.28 - 0.33	3.54 – 3.0	21 - 24	20
Medium silt	33 - 36	0.26 – 0.3	3.85- 3.39	23.25 – 25.5	20
Dense silt	35 - 39	0.23 – 0.27	4.39– 3.69	24.7 – 27.75	20
Very dense silt	38 – 41	0.21 - 0.24	4.81- 4.20	27 – 29.25	20
Natural clays	18 - 32	0.31 – 0.53	3.25- 1.89	12 – 22.5	20
Compacted clays	23 - 37	0.25 – 0.44	4.02- 2.28	15.7 – 26.25	20

We have prepared the range of underground soil data for clays, granular, and intermediate soils with a simple approach which may be used in estimating of outcome of uncertain happening in deterministic method. They are early observations which it is tried to find in past few years. It will be found another observation out of the mentioned ranges possibly. The direct use of the ranges without any tests to investigation for reasons such as lack of layers condition, pre-consolidation, underground water table or relative mix from kind of soils, and rocky case is not logical. Also lack of sufficient awareness of how changing water content, kind of underground soil, relative mix from kind of soils, and lack of layers knowledge in depth are uncertain condition that leads to design and construction in the worst case. The worst case due to uncertain condition could cause unnecessary plan for example more additional depth of retaining wall even more than two times and subsequent problems in construction, mistake in option of appropriate excavation technology and method which are loss of money and time. For that reasons there is need to site geotechnical investigation in logical method.

5.3 Site Geotechnical Investigation

Site investigation is the first activity for exploration of physical, geotechnical, and hydro-geological (if necessary) properties of underground that is base for construction. Physical properties such as specific gravity, particle size and shape (grading), plastic state (Aterberg limits), water or liquid content, and, void ratio of underground could change by spacing and depth. Underground exploration is done by boring, sampling and filling (due to occupancy safety or general safety to avoid dropping hazard). However there are Geophysical tools such as resistivity test, seismic refraction, radar, electromagnetic and gravity technique or aerial geological maps, but these techniques or maps need to be supported by borings for increasing accuracy of the survey, as they cannot give accurately plastic state, shear strength parameters (c', ϕ', ϕ^b), pore water pressure, gradation and soil modulus (e.g. secant, tangent, unloading, reloading, and cyclic modules as a stress-strain relation in different situations).

Site investigation is depending on the essence of the construction project, local complexity, and arduous of the ground condition. As a collective recommendation, ground investigations include considerations about probable suitability (the site due to the projected construction and the rank of tolerable risks) , ground deformation due to construction works, safety due to limit states (any failures), loads relocated from the ground such as lateral pressures, foundation methods (e.g. excavation possibility and stages, sequence of foundation works, excavation supports), influence of construction work on the surroundings, effects of groundwater lowering, and ground capacity to absorb water spewed during construction work [71]. There are minimum requirements in some specifications, recommendations, and building codes about number, spacing, depth of borehole, and quality sampling in countries or different socials. For identification and classification of the ground, at least one borehole or trial pit with sampling should be available. The number of boreholes has to be enough so that with qualified sampling [71], profiles of underground soil(s) or rock and water table could be reliably (statistical hypothesis tests [76]) and economically prepared.

European building code focused on the ground investigation and testing method in Part 2 of Eurocode 7. It includes standards for Laboratory tests in twenty parts of CEN ISO/TS 17892, Sampling and groundwater measurements in EN ISO 22475, Field testing in thirteen part of EN ISO 22476, and identification and classification of soil in EN ISO 14688 [71].

5.3.1 Arrangement of points and depths

The arrangement of points for site investigation should be initially based on area and shape of the structure, critical points of foundation area on base such as corners, and outside of the project land if on a slope. The arrangement of points should not hit vulnerability to the existent structure in site or adjacent structure or road pavement or other public facilities out of site, and the construction work. After initial borings, the primary recognition of underground is observed. Then based on factors such as observed stratification pattern, groundwater monitoring (during the construction period or permanent), load transfer zone of tie-backs if it is in design, and depth extension of several boreholes to all strata which will affect the project or are affected by the project [71] , operation of boring, and sampling could be continued or finished. These factors are assumed base for design of main foundation and retaining wall.

As a rough estimation for initial borings plan for shallow foundation, there have been point's spaces nearly between 10 to 40 meters (33 -131 feet) so that minimum one point for almost each 300 square meters (3200 ft^2) building area and minimum depth about 1.5 to 4 times of foundation width below founding depth in several recommendations [32, 77, 78, 79]. Also there has been appropriately at least one of the bores deeper to 10 meters (30 feet) unless rock-head found and suitably 3 meters (10 feet) below rock-head to demonstrate sound rock in several recommendations [32, 77].

In case of deep excavation, as a rough estimation there have been point's spaces to borings for line of retaining walls between 15 meters to 40 meters (50 -130 ft) spacing located at the wall face with a minimum of one boring at each end of the wall [78]. In several recommendations to boring depths for cantilever walls (e.g., sheet pile, bored pile, diaphragm, and soldier pile) there have been 2 to 2.5 times of excavation depth or bearing stratum or impermeable layer [78, 80] and boring depths for non-cantilever walls (e.g. soil nailing,) as 1 to 1.5 times of excavation depth or bearing stratum or impermeable layer [78, 80].

For different project cases including deep excavation, the minimum requirements of site investigation could be summarized. Separating ground investigation for two separate cases of main foundation and excavation supports could be efficient in detection separate charge and decision-making. Site point's arrangement pattern and depth investigation of high-rise structures with deep excavation based on excavation is indicated in table 5.9 [58]. Table 5.10 shows site points and depth investigation of high-rise structures with deep excavation based on main foundation [58]. Site points and depth investigation of large-area structures with deep excavation based on excavation is represented in table 5.11 [58]. Table 5.12 shows site points and depth investigation of large-area structures with deep excavation based on main foundation [58]. Table 5.13 represents site points and depth investigation of linear structures such as retaining walls, small tunnels with deep excavation based on excavation [58].

Table 5.9: Site points arrangement pattern and depth investigation of high-rise structures with deep excavation based on excavation [58]

Type of structure	**Arrangement pattern**	**Depth of investigation** (D_i) **based on excavations factor:**
High-rise structures with deep excavation	Linear pattern with points at 15 m to 40 m distance located at the wall face	a) If the piezometric surface and the ground-water tables are below the excavation base, the larger value of the following conditions should be met: $D_i \geq 1.4\times h_1$ $D_i \geq (h_1 + t + 2.0)$ m where t is the length of the support penetration under excavation final level and h_1 is the excavation depth b) If the piezometric surface and the ground-water tables are above the excavation base, the larger value of the following conditions should be met: $D_i \geq (h_1 + h_2 + 2.0)$ m $D_i \geq (h_1 + t + 2.0)$ m where h_1 is the excavation depth, h_2 is the height of the groundwater level above the excavation base, and t is the length of support penetration under excavation final level

Table 5.10: Site points and depth investigation of high-rise structures with deep excavation based on main foundation [58]

Type of structure	**Arrangement pattern**	**Depth of investigation** (D_i) **based on main foundation factor:**
High-rise structures with deep excavation	Grid pattern with points at 15 m to 40 m distance	a)Single or strip footing foundation, larger value of the: $D_i \geq$ 6 meters + founding depth $D_i \geq 3.0\times b_f$ + founding depth where b_f is the smaller side length of the foundation b)Raft foundations: $D_i \geq 1.5\times b_b$ + founding depth where b_b is the smaller side of the structure c)Structures with several foundation elements whose effects in deeper strata are superimposed on each other: $D_i \geq 1.5\times b_b$ + founding depth where b_b is the smaller side of the structure d)Piles foundation: $D_i \geq b_b + h_p$+ founding depth $D_i \geq h_p$ +5m + founding depth $D_i \geq h_p +3.0\times d_f$ + founding depth Where d_f is the pile base diameter, b_b is the smaller side of the rectangle circum-scribing the group of piles at the level pile base, and h_p is pile maximum length

Table 5.11: Site points and depth investigation of large-area structures with deep excavation based on excavation [58]

Type of structure	**Arrangement pattern**	**Depth of investigation** (D_i) **based on excavations factor:**
Large-area structures with deep excavation	Linear pattern with points at not more than 60 m distance located at the wall face	a) where the piezometric surface and the ground-water tables are below the excavation base, the larger value of the following conditions should be met: $D_i \geq 1.4\times h_1$ $D_i \geq (h_1 + t + 2.0)$ meters where t is the length of the support penetration under excavation final level and h_1 is the excavation depth b) Where the piezometric surface and the ground-water tables are above the excavation base, the larger value of the following should be met: $D_i \geq (h_1 + h_2 + 2.0)$ meters $D_i \geq (h_1 + t + 2.0)$ meters where h_1 is the excavation depth, h_2 is the height of the groundwater level above the excavation base, and *t* is the length of support penetration

Table 5.12: Site points and depth investigation of large-area structures with deep excavation based on main foundation [58]

Type of structure	**Arrangement pattern**	**Depth of investigation** (D_i) **based on main foundation factor:**
Large-area structures with deep excavation	Grid pattern with points at not more than 60 m distance	Single or strip footing foundation, larger value of: $D_i \geq$ 6 meters + founding depth $D_i \geq 3.0\times b_f$ + founding depth where b_f is the smaller side length of the foundation Raft foundations: $D_i \geq 1.5\times b_b$ + founding depth where b_b is the smaller side of the structure Structures with several foundation elements whose effects in deeper strata are superimposed on each other: $D_i \geq 1.5\times b_b$ + founding depth where b_b is the smaller side of the structure Piles foundation: $D_i \geq b_b + h_p$+ founding depth $D_i \geq h_p$ +5m + founding depth $D_i \geq h_p + 3.0\times d_f$ + founding depth Where d_f is the pile base diameter, b_b is the smaller side of the rectangle circum-scribing the group of piles at the level of the pile base, h_p is pile maximum length

Table 5.13: Site points and depth investigation of linear structures such as retaining walls, small tunnels with deep excavation based on excavation [58]

Type of structure	**Arrangement pattern**	**Depth of investigation** (D_i) **based on excavations factor:**
linear structures with deep excavation such as retaining walls, small tunnels	Spacing of 20 m to 200 m between points located at the wall face	a) where the piezometric surface and the ground-water tables are below the excavation base, the larger value of the following conditions should be met: $D_i \geq 1.4\times h_1$ $D_i \geq (h_1 + t + 2.0)$ m where t is the length of the support penetration under excavation final level and h_1 is the excavation depth b) Where the piezometric surface and the ground-water tables are above the excavation base, the larger value of the following conditions should be met: $D_i \geq (h_1 + h_2 + 2.0)$ m $D_i \geq (h_1 + t + 2.0)$ m where h_1 is the excavation depth, h_2 is the height of the groundwater level above the excavation base, and *t* is the length of support penetration under excavation final level c) small tunnels: $b_a + h_e < D_i < h_e + 2\, b_a$ where b_a and h_e are excavation width and final depth

5.3.2 Sampling

In each boring, samples should be obtained from each separate ground layer. It is assumed that soil samples for laboratory tests are remain unaffected during sampling, handling, transport, and storage. This is because of water content, density, permeability, compressibility, particle sizes, and shear strength of soil. Shear strength and consolidation samples must be undisturbed. Sampling may be replaced by several simple field tests such as soil penetration test (SPT) or cone penetration test (CPT) if there is adequate knowledge to equalize or correlate the field tests with the ground conditions.

Boring may be done by truck-mounted auger boring, wash boring, core boring and/or test pits. Core boring is used to rock sampling in common. Test pits are relatively rapid, inexpensive, easy and reliable in site testing and sampling, and it is suitable to taking undisturbed samples. Auger boring has difficulties in very soft clay, or coarse sand. Instead of drilling, sometimes backhoes may be enough in lower depths. A truck-mounted rotary drilling rig machine could bore holes up to 200 mm (eight inches) in diameter, and 200 meters (656 feet) deep. A truck-mounted drilling rig is shown in figure 5.2. Typical truck-mounted rotary drilling data is indicated in table 5.14 [18].

Table 5.14: Typical truck-mounted rotary drilling rig data [18]

Depth (meters)	Up to 500
Diameter (inch)	6.5 – 8.5
Mast capacity (ton)	18-25
Mast height (meters)	8-9
Rod handling capacity	6.1 m (20 feet)
Rotary spindle (rpm)	0 – 200
Feed cylinder capacity (mm×mm)	125×2350 - 160×2350
Maximum load lifting (kg)	7250 -12000
Compressor 1100cfm pressure (psi)	300-350
Maximum pull up speed (m/sec)	0.5- 0.75
Maximum pull in speed (m/sec)	0.75-1.2

Figure 5.2: Truck mounted drilling rig [18]

5.3.3 Water table

Groundwater table and permeability of total underground soil could be determined by minimum 3 wells (prefer 2 minors well in a same direct from main) and one pump test. Permeability of ground is obtained by [77]:

$$k = Q\times \text{Ln}\ [(r_2/r_1) / [\pi \times (h_2^2 - h_1^2)]] \qquad \text{(Eq 5.1)}$$

Where Q = water discharge by pump, r_1 and r_2 are distance of other two minor wells from pumping well, h_1, and h_2 are increased level of water relative to main pumped well water level in two minor wells (corresponding h_1 and h_2) after 24 hours of pumping that is shown in figure 5.3.

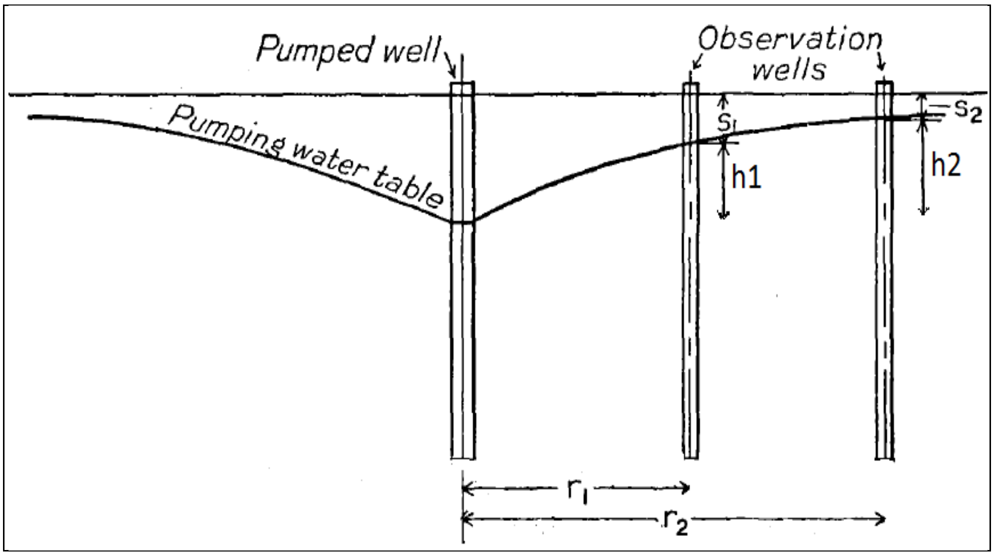

Figure 5.3: The wells measurements for estimating ground permeability [77]

5.3.4 Characteristic property

Characteristic property ($P_{charactristic}$) is simply derived by decreasing the half of standard deviation from arithmetic mean (S_{mean}) of samples property (Schnider 1999):

$P_{charactristic} = y_{mean} - 0.5\times$ [standard deviation] (Eq 5.2)

Where standard deviation = $[\,[\,\Sigma\,(y_i - y_{mean})^2\,]\,/\,(n\,-1)\,]^{0.5}$

The y_i is each observation; the y_{mean} is the mean of the total observations, and the n is the number of replications [76, 81].

Due to importance of shear strength and oedometer modulus E_{oed} for engineering estimates, Eurocode 7 has been giving some recommendation about minimum number of their observations. Minimum number of triaxial test to determine the effective angle of shearing resistance is indicated in table 5.15. The recommendation for the required number of triaxial tests to determine undrained shear strength is indicated in table 5.16. One recommended test defines a set of three individual specimens tested at different cell pressures [71]. The recommendation for the required number of oedometer tests to determine modulus E_{oed} is shown in table 5.17 [58]. One recommended test defines a set of three individual specimens tested at different cell pressures [58]. It seems the above recommendations are due to relatively significance differences in observations which can be source of risk.

Table 5.15: Number of triaxial tests (1 test = 3 specimens tested) to determine the effective angle of shearing resistance [58]

Variability in strength envelope coefficient of correlation r on regression curve	**Comparable experience (None)**	**Comparable experience (Medium)**	**Comparable experience (Extensive)**
$r \leq 0.95$	4	3	2
$0.95 \leq r \leq 0.98$	3	2	1
$r \geq 0.98$	2	1	1

Table 5.16: Number of triaxial tests (1 test = 3 specimens tested) to determine undrained shear strength [58]

Variability in strength envelope coefficient of correlation r on regression curve	**Comparable experience (None)**	**Comparable experience (Medium)**	**Comparable experience (Extensive)**
Maximum value / minimum value ≥ 2	6	4	3
$1.25 \leq$ Maximum value / minimum value ≤ 2	4	3	2
Maximum value / minimum value ≤ 1.25	3	2	1

Table 5.17: Recommendation for number of oedometer tests to determine modulus E_{oed} [58]

Variability in oedometer modulus E_{oed} (in the related stress range)	**Comparable experience (None)**	**Comparable experience (Medium)**	**Comparable experience (Extensive)**
Range of values of $E_{oed} \geq 50$ %	4	3	2
20 % < Range of values of E_{oed} < 50 %	3	2	2
Range of values of $E_{oed} \leq 20$ %	2	2	1

5.3.5 Cost of site investigation

Site investigation and its costs are requiring for design and construction of building foundation. It could be used in deep excavation supporting system design and implementation if the Site investigation plan has been suitable. A suitable plan for testing could be approximately \$100 per hour for specialist to planning plus indirect costs such as costs for transportation, visit the site and other requirements. As an initial percentage of ordinary buildings project estimated costs, foundation costs are estimated between 0.5% to 2% and site investigation costs are estimated between 0.05% to 0.2% project total costs by ordinary estimators [34] however not only it is differing in relative location (country, region) but also projects involved deep excavation are not ordinary buildings. For volume around 131000 m^3 favorable (sandy lean clay with gravel in contrast of CL soil type) soil excavation with depth of 20 meters in Lefkosha (Nicosia) in North Cyprus, the direct cost of each one cubic meters of deep excavation has more than \$16.15 against \$3.5 (normal excavation) . The direct cost of each one cubic meters of deep excavation was more than four times and more than 9-fold increase in time of ordinary excavation [82]. Also the sum of direct cost of excavation and foundation were more than 11% of total building cost for the 50750 m^2 building area with five underground basements, ground floor, and 7 stories. The deep excavation implemented from Mars 2015 until January 2016. The cross-section of the project is shown in figure 5.4. The project is geotechnical investigated by four boreholes jointly with SPT in situ test, and 45 sampling for laboratory tests of: 4×(Plastic, and Liquid limits), 5×(water content), 2×(cohesion), 1×(internal friction degree ϕ), 2×(unit weight γ), and 45×(grading, sieve analysis, clay content, percent fines). Cost of those tests is less than about \$4100 for laboratory tests and less than about \$2500 for boring and sampling which sum is $6600. All those tests were requiring for main foundation design and actually there wasn't any cost for deep excavation. It is possible that site investigation ordinary estimated costs cannot cover the necessary costs to show the contrasts due to difference in underground conditions.

Borehole costs include supply rig onto site (on site), set-up on each new hole (per hole), and hole drilling (per meter). Approximate ranges of costs of boreholes in Great Britain are indicated in table 5.18 [77]. It should be noted that for international contracts exchange rates are added on costs. Cost of onsite is increased rapidly in depriver, remote, or insecure areas.

Laboratory tests are distributed as index tests, strength and strain tests, consolidation tests, permeability tests, and material tests. Index test includes moisture content, grading (sieve analysis, percent fines, and hydrometer analysis), shrinkage and liquid and plastic limits (Atterberg), specific gravity, natural density, and proctor density. The important laboratory testing with ASTM standards for deep excavation are gathered that indicated in table 5.19 [80].

The important in situ and field testing with ASTM standards for deep excavation are gathered that shown in table 5.20 [80]. The unit prices of laboratory tests are commonly based on the samples delivered to laboratory. Testing prices are direct costs, thus indirect costs are added them. After required tests and tests report, an experienced geotechnical engineer (individual person or a team) studies the tests report and prepares underground profiles with hypothesis testing of observations [76] that may lead to other additional boring, laboratory, and/or field tests to reach an appropriate reliability. Then technical alternatives based on existence reliable exploration are preliminarily designed.

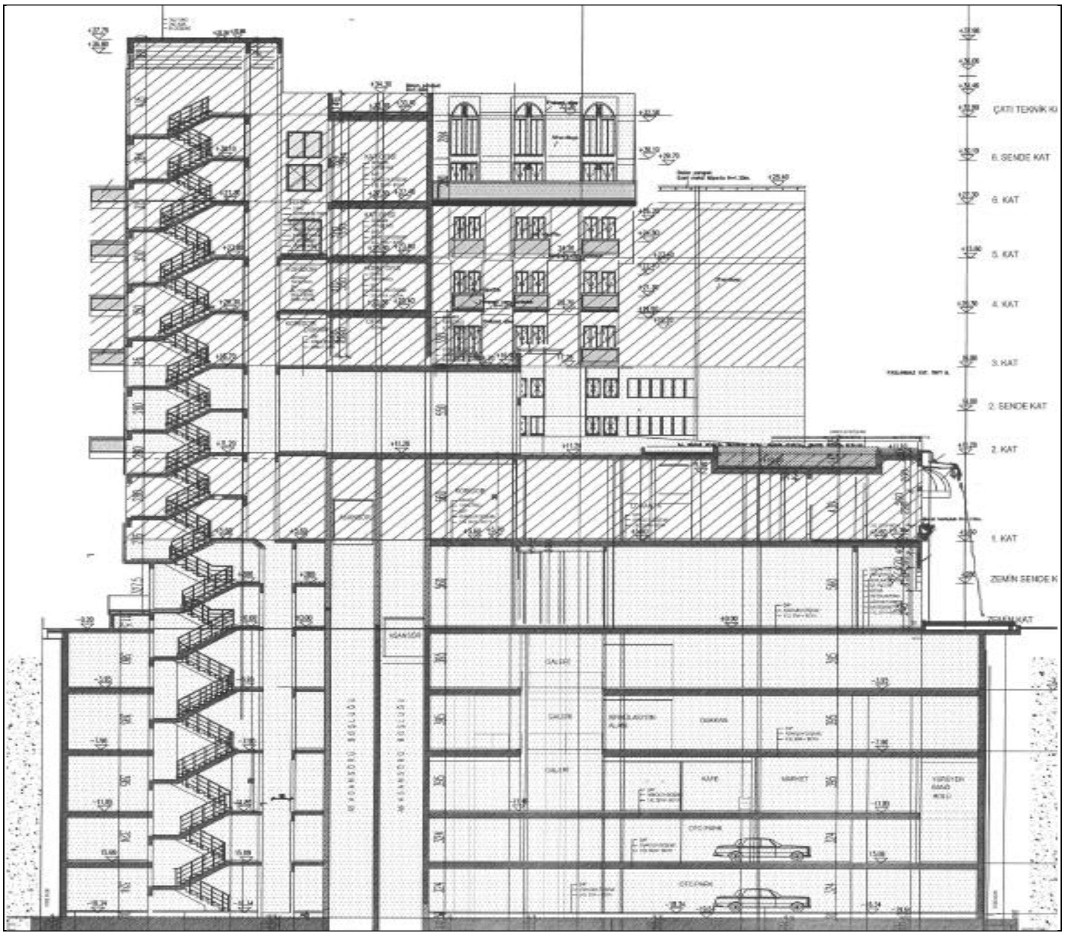

Figure 5.4: The cross-section of a case study in lefkosha (Nicosia) Northern Cyprus

Table 5.18: Approximate ranges for cost of boreholes in Great Britain (USD)

Case [77]	**On site** ($)	**Per hole** ($)	**Per meter** ($)
Light percussions, soil< 10 meters deep	202-540	32-82	11- 27
Light percussions, soil> 10 meters deep	202-540	32-82	16- 41
Probing in rock or soil	308- 820	11-26	11-26
Rotator coring in rock	412-1100	41- 109	41-110
Trial pits , 4 meters deep , backfilled	$155- 412 for 3 pits		

Table 5.19: Laboratory testing standards (ASTM) required for deep excavation

Laboratory Tests	**Standard** [80]	**Unit**
INDEX TESTS		
Moisture Content	ASTM D 2216	Each
Percent Fines (passing #200)	ASTM D 1140	Each
Sieve Analysis, Clay content	ASTM D 422	Each
Hydrometer Analysis	ASTM D 422	Each
Shrinkage, Liquid, and Plastic Limits	ASTM D 4318, D4038, D427	Each
Natural Density	ASTM D 698, D 1557	Each
Specific Gravity	ASTM D 854	Each
Proctor Density	ASTM D 698 or D 1557	Each
STRENGTH & or STRAIN TESTS		
Soil Unconfined Compressive Strength (remolded)	ASTM D 2166	Each
Soil Unconfined Compressive Strength (Shelby)	ASTM D 2166	Each
Rock Unconfined Compression strength	ASTM D 2938	Each
Drained Direct Shear (Peak)	ASTM D 3080	Each
Direct Shear(additional points over 3)	ASTM D 3080	Point
Triaxial Shear (CU)	ASTM D 2850, D 4767	Each
Triaxial Shear (UU)	ASTM D 2850, D 4767	Each
Triaxial Shear (CD)	ASTM D 2850, D 4767	Each
Vane Shear for Saturated Clayey Soil	ASTM D 4648	Each
CONSOLIDATION TESTS		
Consolidation (11 points with rebound)	ASTM D 2435	Each
Additional Load over 11 points	ASTM D 2435	Point
Additional Unload-Reload Cycle	ASTM D 2435	Each
Swell and Swell Pressure	ASTM D 4546	Each
PERMEABILITY and SUCTION TESTS		
Rigid Wall Permeability	ASTM D 5856	Each
Filter paper	ASTM D 5298	Each
MATERIAL TESTS		
Rock Compressive Strength	ASTM D 7012-C	Cylinder
Concrete Compressive Strength	ASTM C 40	Cylinder
Compressive Strength of Drilled Cores	ASTM C 42/C 42M-13	Each
Grout Compressive Strength (2" Cubes)	ASTM C109	Cube
Preparation of Specimen (Sample Pickup, Capping, Trimming, etc.)	ASTM D 1586, D 1587	Hour
Coefficient of Friction,	ASTM D 5183	Each
Density of Bentonitic slurries	ASTM D 4380	Each
Slurry seal	ASTM D 3910	Each

Table 5.20: In situ activities or field testing according to standards (ASTM)

Field Tests	**Standard** [80]	**Unit**
Rock core drilling	ASTM D 2113-14	Each
Bulk Soil Sample Handling (> 5 lbs)	ASTM C 999	Each
California Bearing Ratio (3 point)	ASTM D 1883	Each
California Bearing Ratio (per additional point)	ASTM D 1883	Each
Residual Direct Shear	ASTM D 4554 - 12	Each
Vane Shear for Saturated Clayey Soil	ASTM D 2573	
Stiffness and Apparent Modulus of Soil (Electro-Mechanical method)	ASTM D 6758 - 08	Each
Rigid Plate loading	ASTM D 4394-08	Each
flexible Plate loading	ASTM D 4395-08	Each
Standard penetration test (SPT)	ASTM D 1586	Each
Pressure meter	ASTM D 4719	Each
Flat plate Dilatometer	ASTM D 6635	Each
Vertical Inclinometer probe	ASTM D 7299-12	Each
Rolling Inclinometer	ASTM E 2133 - 03	Each
Visual Identification	ASTM D 2488	Each
Ground Penetrating Radar (No Mobilization Fee)	ASTM D 6432	Day
Electrical Resistivity Test	ASTM G 57	Each
Monitoring Well	ASTM D 5092, D 4750	Each
Field Permeability	ASTM F 2898-11	Each
Temporary Casing (50-150 cm diameter)	ASTM D 5876-95	L.Foot
Ground movement-probe inclinometer	ASTM D 6230	Each
Pore water extraction	ASTM D 4542	Each
Liquefaction potential	ASTM D 6066	Each
Water leakage of wall	ASTM E 2128	Each
Axial load of deep foundation	ASTM D 1143	Each
Flow rate of water and slurry	ASTM D 7701	Each
Cone penetration (CPT)	ASTM D 217	Each
Piezocone penetrometer	ASTM D 6067, D 5778	Each
Shelby Tube Extrusion		Each
Mob/demob of drill		Hour
Specific Storage coefficient (dewatering)	ASTM D4630, D5270, D5850, D4106, D4105	Each

5.4 Conclusion

Underground soil may be source of risk for deep excavation which identification needs to special classification based on range of properties for each parameter in order to risk identification. In that manner a general special classification of soils is proposed based on early experiences that divides underground soils to clays in five main group, granular soils (such as gravel, sand, and silt) in five main groups, and intermediate soils which are combination of clays and granular soils.

Site geotechnical investigation is overviewed which includes arrangement of points based on required spacing and depth, sampling from each required depth of each borehole with related technology, characteristic property of each soil parameter which is obtained by statistic analysis of different observations which may be had significant differences for design of main foundation and deep excavation supports but may not enough for all probable construction situations (see illustrated example 9.7), and cost of site investigation which include plan to arrangement of points and minimum required sampling, boring costs and a totally tests cost for a case studied in Northern Cyprus.

The required standard laboratory and in-situ field test methods in deep excavation are collected which may be effective in test orders and cost control.

There are important recommendations about number of minimum required tests for shear strength and oedometer modulus because of serious differences in a ground soil property.

The cost of geotechnical investigation tests is approximately between 0.0002 to 0.00024 of total building construction costs in spring mall project scale which is deterministic in design and construction of deep excavation.

Chapter 6

GROUND AND SUPPORTS FAILURES

6.1 Introduction

There are questions such as what are ground failure, why, when and how it occurs. However soil classification based on type and range of parameters of each type was done, or site investigation is finished but there isn’t any sign of risk damage until vertical cutting is made and consequence of inclining to lateral expand and shift into the scratch zone is occurred which is a mode of underground failure. Underground failure may be assume risk source which is identifies risk and is function of soil type and parameters which is investigated and classified.

Results of inclining to lateral expand and shift into the scratch zone is obviously existence of un-equilibrium in stresses and huge strains in ground which is cut. The variation of moisture content of soil (as a unit solid) or internal water table could affect internal stresses or strains and subsequently influence the stability of an excavation vertical surface. A small displacement in soil (as a solid) leads to conceptual shear stress so that in case of failure converts to shear strength. Factors affected on the shear strength of soil may be composition of soil (mineralogy, grain size distribution and shape of particles of the soil mass), the structure of soil layers (arrangement of soil particles and soil layers and joints), and initial soil state (the stress history of the soil) [66, 67, 70].

Shear strength in case of unsaturated soils (above the ground water table) could be identified and predicted by extended Mohr-Columb failure criterion [83, 84]:

$\tau_{strength} = c' + (\sigma_n - u_a)\tan\phi' + \chi\,(u_a - u_w)$ (Eq 6.1)

where c'= effective cohesion, $\chi = \tan\phi^b$, ϕ^b = friction angle associated with the suction, σ_n = vertical effective stress = vertical total stress - pore water pressure, u_a = pore air pressure, ϕ'= effective internal friction angle, u_w = pore water pressure.

There are empirical formulas by Oberg and Sallfors for sands and silts [85]:

$\chi = S_r \times \tan\phi'$ (Eq 6.2)

where S_r is degree of saturation. [$S_r = w \times G/e$, G is specific gravity)]

Another empirical research was proposed by Kayadelen et al (2007), for clays between degree of saturation and χ factor that are shown in figure 6.1[85].

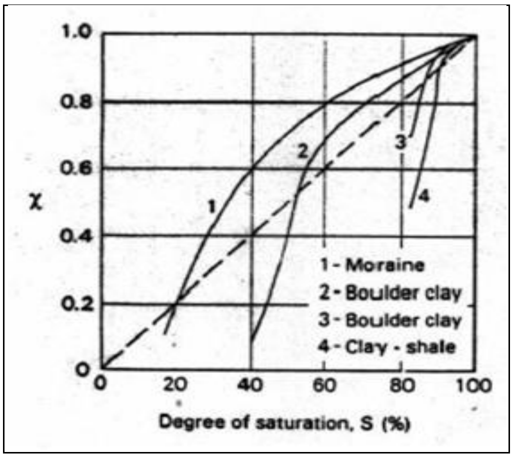

Figure 6.1: The χ factor vs. degree of saturation for clays [82]

Also the χ-factor was proposed by Khalili and Khabbaz (1998) as:

$\chi = [(u_a - u_w) / (u_a - u_w)_b]^{-0.55}$ (Eq 6.3) where $(u_a - u_w)_b$ is the air entry value of soils that obtained by soil water characteristic curve (SWCC) test.

Vanapalli (1996) proposed another relationship to predicting χ that is:

$\chi = \tan\phi^b = (\theta_w / \theta_s) \times \kappa \times \tan\phi'$ (Eq 6.4)

where θ_w = volumetric water content (from the compaction curve), κ = fitting parameter, θ_s = saturated volumetric water content (from the soil water characteristic curve), ϕ' = friction angle (from direct shear test on saturated sample),

$\kappa = 0.98 + 0.0874 \times I_p - 0.001 \times (I_p)^2$ (I_p = plasticity index)

Also there is proposed relationship between ϕ^b vs. water content (w) [85]:

$\phi^b = 1.26 \times w - 17.63$ (Eq 6.5)

In case of saturated soils, $(u_a - u_w)$ is zero and shear strength is estimated as:

$\tau_{strength} = c' + (\sigma_n - u_a) \tan\phi'$ (Eq 6.6)

The angle between horizontal plan and shear failure plan in triaxial or odometer tests is observed about $45^o + \frac{\phi}{2}$ and $\sin\phi = (\sigma_1' - \sigma_2') / (\sigma_1' + \sigma_2')$ in failure moments [77]. Figure 6.2 shows shear strength vs. effective normal stress in a typical saturated soil [84].

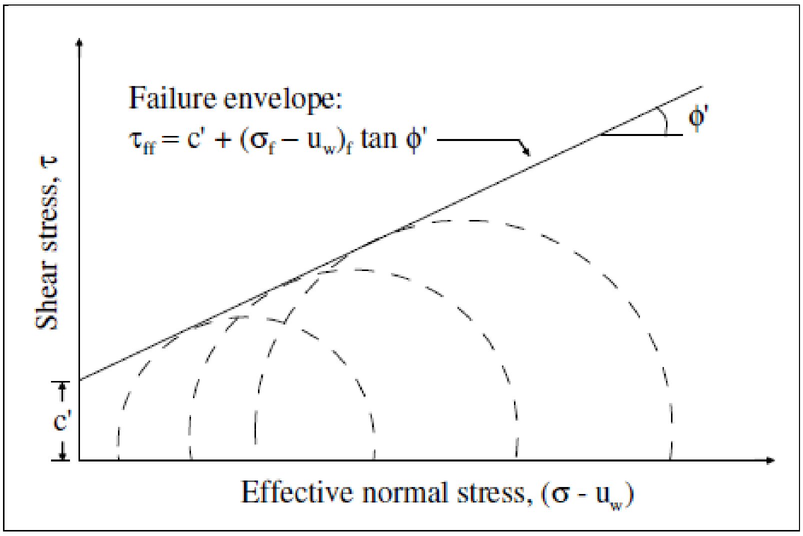

Figure 6.2: Typical Mohr-Columb failure envelopes for a saturated soil [84]

The zone above the water table is divided into three separate sections which from down to up and with arrangement they are capillary fringe (solid + water), two fluid phases (solid + water + air), and dry soil (solid + air) [80, 81]. There is negative pore water pressure (less than atmospheric value) in capillary fringe. The height of capillary rise in different soils is indicated in table 6.1 [77]. As it is seen in clays height of capillary rise is more than 10.0 meters while in granular soils it is between zeros to 3.50 meters. Also in silty soils it is between 1.5 to 12 meters.

Table 6.1: The height of capillary rise in different soils [77]

Soil	**The height of capillary rise**
Open graded gravel	0
Coarse sand	0.03 – 0.15 m (0.1 – 0.5 ft)
Medium sand	0.12 – 1.1 m (0.4 – 3.6 ft)
Fine sand	0.30 – 3.5 m (1.0 – 12.0 ft)
Silt	1.5 – 12.0 m (5.0 – 40.0 ft)
Clay	More than 10.0 m (more than 33.0 ft)

The water content of saturated soil leads to drained (c, ϕ) and undrained (c', ϕ') shear strength concepts and differences. Undrained shear strength of soil is identified based on effective stress from the soil that water can't go to out or flow into the soil (undisturbed sampling). Drained shear strength of soil is identified based on total stress from the soil by emptying pore water so that there isn't water (disturbed sampling). The relationship between total stress and effective stress is calculated by:

Total stress (σ) = Effective stress (σ') + Pore water pressure (u) (Eq 6.7)

The quantity of total stress are calculated as a function of depth by accumulating unit weight of the soil ($\gamma_{soil} \times h$) and the effective stress as a function of depth by accumulating dry unit weight of the soil ($\gamma_{dry} \times h$) and Pore water pressure as a function of depth by accumulating unit weight of water ($\gamma_w \times h$).

Stability of retaining wall may be affected by sliding along the base due to excessive lateral loads, overturning due to excessive moment of lateral loads, bearing capacity due to base failure, excessive settlement due to below weak soil layer, deep seated shear failure, and a few known failure modes which are gathered and presented in this chapter. Although EC7 is based on limit states and defines partial coefficients for actions and resistant instead of factor of safety for design, but it is for design and for the reason of its simplified using for risk assessment in deep excavation construction based on description of risk analyzing in chapter 3, factor of safety (FOS) is used for each case of failure analyzing.

6.2 Sliding failure

When sliding occurs the sided wall of excavation is collapsed and supporting system, machines, human, and all things that are in the crosshairs could be probably buried. Also the near urban facilities, and adjacent buildings with shallow foundation in distance less than or maximum equal to depth of excavation (in short term) could be cracked, settled, and even collapsed. Figure 6.3 shows the common sliding surface in vertical excavation and its incline of shearing is observed in triaxial or oedometer test. For surveying and preventing the sliding failure, the resistance to sliding/ horizontal component of resultant lateral earth pressure ratio ($\sum F_R$ / $\sum F_S$) is calculated. The $\sum F_R$ is being sum of resistant forces against sliding such as passive force, friction force between wall and soil, and adhesion of wall footing on soil. The $\sum F_S$ is being sum of active forces. Friction force between wall and soil is multiplication of resultant of all active forces (R) that act on retaining wall to friction coefficient ($\tan\delta$) [66, 67]. Simplified distribution of earth pressure is shown in figure 5.4. Adhesion is function of effective width of wall on soil (C_b B). Adhesion of footing of concrete wall is function of soil cohesion as could be calculated by table 6.2 [66, 67]. Factor of safety (FOS) against sliding is defined as [86]:

$$FOS = \sum F_R / \sum F_S \qquad \text{(Eq 6.8)}$$

where $\sum F_R$ = Resistance to sliding = $R \tan \delta + C_b B + \sum P_{passive}$

$\sum F_S = \sum P_{active}$

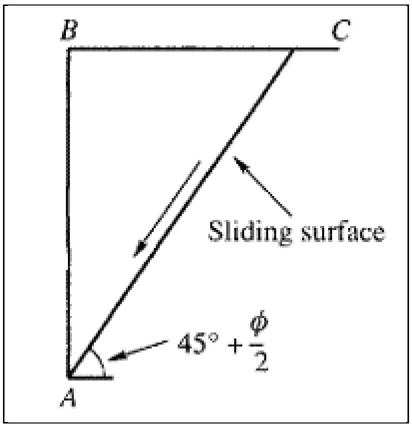

Figure 6.3: Typical sliding failure surfaces [66, 67]

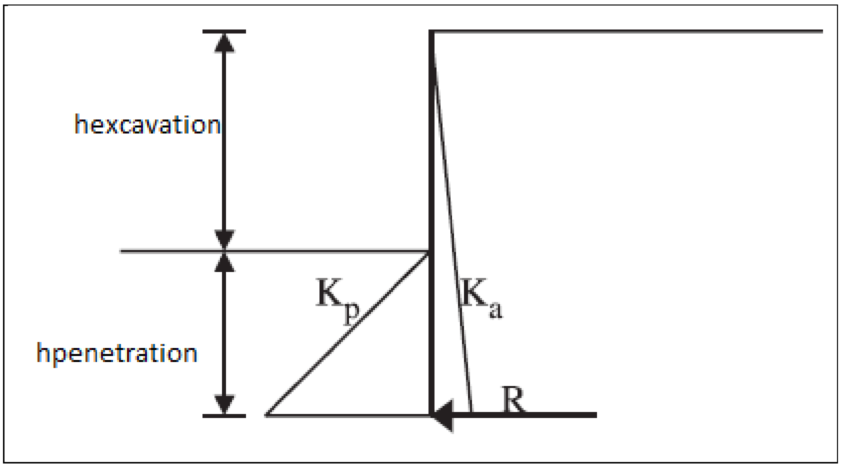

Figure 6.4: Simplified distribution of earth pressure (without surcharge) [66, 67]

Table 6.2: Adhesion for concrete on soil based on soil cohesion [66, 67]

c_u (soil cohesion)	C_b (adhesion for concrete on soil)
23.9 kN/m^2 (500 lb/ft^2) or less	$C_b = c_u$
23.9 - 47.9 kN/m^2	$C_b = 0.5\times c_u + 13.92$ (kN/m^2)
Equal 47.9 kN/m^2 (1000 lb/ft^2)	$C_b = 0.75\times c_u$
47.9 – 95.8 kN/m^2	$C_b = 0.25\times c_u + 23.96$ (kN/m^2)
Equal 95.8 kN/m^2 (2000 lb/ft^2)	$C_b = 0.5\times c_u$
95.8 – 191.5 kN/m^2	$C_b = 0.16\times c_u + 32.58$ (kN/m^2)
>191.5 kN/m^2(4000 lb/ft^2)	$C_b = 0.33\times c_u$

If there is one tip of soil, the sum of passive forces could be calculated by (Bell):

$$\sum P_{passive} = 0.5\times \gamma_{soil} \times h^2 \times K_p + 2\times c_u\times h \times K_p^{0.5} \qquad \text{(Eq 6.9)}$$

In this case the sum of active forces could be calculated by (Bell):

$$\sum P_{active} = q_{surcharge} \times K_a\times h + 0.5\times\gamma_{soil} \times h^2\times K_a - 2\times c_u\times h\times K_a^{0.5} \qquad \text{(Eq 6.10)}$$

If there are two layers of soils, the sum of passive forces could be calculated by (Bell):

$$\sum P_{passive}=0.5\times\gamma_{soil1}\times h_1^2\times K_{p1}+2\times c_{u1}\times h_1\times K_{p1}^{0.5}+0.5\times(\gamma_{soil1}\times h_1+\gamma_{soil2}\times h^2)\times h^2 \times K_{p2}$$

In this case the sum of active forces could be calculated by (Bell):

$$\sum P_{active} = q_{surcharge}\times K_{a1}\times h_1+0.5\times\gamma_{soil1}\times h_1^2 \times K_{a1}- 2\times c_{u1}\times h_1\times K_{a1}^{0.5}+ \\ + 0.5\times (\gamma_{soil1}\times h_1+\gamma_{soil2}\times h_2)\times h_2\times K_{a2}+ q_{surcharge} \times K_{a2}\times h_2$$

If there are n layers of soils, the sum of passive forces could be calculated by (Bell):

$$\sum P_{passive} = 2\times c_{u1}\times h_1 \times K_{p1}^{0.5} + \sum [(0.5\times h_j\times K_{pj}) \times\sum (\gamma_i\times h_i)]$$

In this case the sum of active forces could be calculated (Bell) by:

$$\sum P_{active} = [q_{surcharge}\times\sum (K_{aj}\times h_j)] - 2\times c_{u1}\times h_1\times K_{a1}^{0.5}+ \sum [(0.5\times h_j\times K_{pj}) \times\sum (\gamma_i\times h_i)]$$

Where $j = 1, 2, \ldots, n$ are number of up layers to reach in layer number n, and $i = 1, 2, \ldots, j$ are number of up layers to reach in layer number j (final later).

The range of active and passive forces of soil and collect them in a table to speed up the calculation in field for checking the sliding stability in pre excavation without any supports, checking the depth of each stage of excavation, and design could be necessary that they and tanδ for dense sand, dry loose sand, stiff clays, and soft clays is calculated and indicated in tables 6.3, 6.4, and 6.5 respectively. The H in those tables is wall length. Wall length is equal to sum of excavation final depth (hexcavation) and penetration of wall in ground under final level of excavation (h). The penetration of wall in ground under final level of excavation (h) can produce the passive force.

$H = H_{wall} = h_{excavation} + h_{penetration}$

Annex C of EN 1997-1 gives expressions for active and passive earth forces:

$\sum P_{active} = q_{surcharge} \times h \times K_a - K_a \times u \times h + 0.5 \times \gamma_{soil} \times h^2 \times K_a + u \times h - 2 \times c_u \times h[K_a(1+ C_b / c_u)]^{0.5}$

$\sum P_{passive} = q_{surcharge} \times h \times K_p - K_p \times u \times h + 0.5 \times \gamma_{soil} \times h^2 \times K_p + u \times h + 2 \times c_u \times h[K_p(1+ C_b / c_u)]^{0.5}$

where u is pore water pressure, γsoil is weight density of retained soil.

The difference between Bell and recent formulas is on coefficient of $(1+ C_b / c_u)^{0.5}$ in final statement. In soft clays its impact on final statement is 1.338 and in hard clays is 1.153 while the Bells method is simple actually.

Table 6.3: The range of soil active and passive forces, tanδ for dense sand and dry loose sand and range of external friction coefficient between concrete and soil (Bell's method)

γ_{soil} , K_a	**Soil active force**	**Soil passive force**	**tanδ, concrete**
Dense sand			
17.2,0.198	19.823H +1.704778H^2	43.3839h^2	0.509525
17.2,0.198	19.823H +1.704778H^2	43.3839h^2	0.57735
17.2,0.238	21.78831H+2.045795H^2	36.152207 h^2	0.509525
17.2,0.238	21.78831H+2.045795H^2	36.152207 h^2	0.57735
18.3,0.198	19.823H +1.8138045H^2	46.158456 h^2	0.509525
18.3,0.198	19.823H +1.8138045H^2	46.158456 h^2	0.57735
18.3,0.238	19.823H +2.1766304 H^2	38.4642667 h^2	0.509525
18.3,0.238	19.823H +2.1766304 H^2	38.4642667 h^2	0.57735
Loose sand			
14, 0.2486	24.86H+1.74 H^2-145.59H	28.16h^2+585.642h	0.4296339
14, 0.2486	24.86H+1.74 H^2-145.59H	28.16 h^2+585.642h	0.4931454
14, 0.295	29.5H+2.065 H^2- 158.597H	23.728 h^2+537.615h	0.4296339
14, 0.295	29.5H+2.065H^2- 158.597H	23.728 h^2+537.615h	0.4931454
15.6, 0.249	24.86H+1.94 H^2-158.597H	31.3757 h^2+585.642h	0.4296339
15.6, 0.249	24.86H+1.94 H^2-158.597H	31.3757 h^2+585.642h	0.4931454
15.6, 0.295	29.5H+2.3 H^2-158.597H	26.441 h^2+537.6155h	0.4296339
15.6, 0.295	29.5H+2.301 H^2-158.597H	26.4406 h^2+537.615h	0.4931454

Table 6.4: The ranges of soil active and passive force, tanδ for Stiff clays (by groundwater level) and range of external friction coefficient between concrete and soil (Bell's method)

γ_{soil} , c_u , K_a	**Soil active force**	**Soil passive force**	**tanδ (concrete)**
16.9, 48, 0.2	20H+1.69H^2- 42.93H	42.25h^2+214.6625258h	0.3249197
16.9, 48, 0.2	20H+1.69H^2- 42.93H	42.25h^2+214.6625258h	0.624869
16.9, 48, 0.5	50H+4.23H^2- 67.88H	16.9h^2+135.7645h	0.3249197
16.9, 48, 0.5	50H+4.23H^2- 67.88H	16.9h^2+135.7645h	0.624869
16.9, 96, 0.2	20H+1.69H^2- 85.86H	42.25h^2+429.3250516h	0.3249197
16.9, 96, 0.2	20H+1.69H^2- 85.86H	42.25h^2+429.3250516h	0.624869
16.9, 96, 0.5	50H+4.23H^2-135.76H	16.9h^2+271.529h	0.3249197
16.9, 96, 0.5	50H+4.23H^2-135.76H	16.9h^2+271.529h	0.624869
19, 48, 0.2	20H+1.9 H^2-42.932H	47.5h^2+214.6625258h	0.3249197
19, 48, 0.2	20H+1.9 H^2-42.932H	47.5h^2+214.6625258h	0.624869
19, 48, 0.5	50H+4.75 H^2-67.88H	19 h^2+135.7645h	0.3249197
19, 48, 0.5	50H+4.75 H^2-67.88H	19 h^2+135.7645h	0.624869
19, 96, 0.2	20H+1.9 H^2-85.865H	47.5h^2+429.3250516h	0.3249197
19, 96, 0.2	20H+1.9 H^2-85.865H	47.5h^2+429.3250516h	0.624869
19, 96, 0.5	50H+4.75 H^2- 135.7H	19 h^2+ 271.529h	0.3249197
19, 96, 0.5	50H+4.75 H^2- 135.7H	19 h^2+ 271.529h	0.624869

Table 6.5: The ranges of soil active and passive force, tanδ for Soft clays (by groundwater level) and range of external friction coefficient between concrete and soil (Bell's method)

γ_{soil} , c_u , K_a	**Soil active force**	**Soil passive force**	**tanδ (concrete)**
15.6, 12, 0.5	$50H+3.9H^2- 16.97H$	$9.75h^2+26.832815h$	0.50777
15.6, 12, 0.5	$50H+3.9H^2- 16.97H$	$9.75h^2+26.832815h$	0.55774
15.6, 12, 0.8	$80H+6.24H^2- 21.46H$	$15.6h^2+33.941125h$	0.50777
15.6, 12, 0.8	$80H+6.24H^2- 21.46H$	$15.6h^2+33.941125h$	0.55774
15.6, 24, 0.5	$50H+3.9H^2- 33.941H$	$9.75 h^2+53.665631h$	0.50777
15.6, 24, 0.5	$50H+3.9H^2- 33.941H$	$9.75 h^2+53.665631h$	0.55774
15.6, 24, 0.8	$80H+6.24H^2- 42.93H$	$15.6h^2+67.88225h$	0.50777
15.6, 24, 0.8	$80H+6.24H^2- 42.93H$	$15.6h^2+67.88225h$	0.55774
17.8, 12, 0.5	$50H+4.38 H^2-16.97H$	$11.125h^2+26.83281h$	0.50777
17.8, 12, 0.5	$50H+4.38 H^2-16.97H$	$11.125h^2+26.83281h$	0.55774
17.8, 12, 0.8	$80H+7.12 H^2-21.46H$	$17.8 h^2+33.941125h$	0.50777
17.8, 12, 0.8	$80H+7.12 H^2-21.46H$	$17.8 h^2+33.941125h$	0.55774
17.8, 24, 0.5	$50H+4.45 H^2-33.94H$	$11.125h^2+53.665631h$	0.50777
17.8, 24, 0.5	$50H+4.45 H^2-33.94H$	$11.125h^2+53.665631h$	0.55774
17.8, 24, 0.8	$80H+7.12 H^2-42.93H$	$17.8 h^2+67.88225h$	0.50777
17.8, 24, 0.8	$80H+7.12 H^2-42.93H$	$17.8 h^2+67.88225h$	0.55774

6.3 Overturning

Excessive lateral earth pressures due to actions with relation to retaining wall resistance cause the retaining wall system to topple or rotate (overturning) failure. If $\sum M_R$ is sum of resistant moments and $\sum M_O$ is sum of overturning moments according to mechanical principles, then the relation of $\sum M_R$ / $\sum M_O$ is overturning factor of safety (FOS) against overturning[66, 67, 70]. The overturning moment is calculated as:

$$\begin{aligned}\sum Mo= &(1/6)\, \gamma'_{soil}\times K_a \times (h_{excavation} + h_{penetration})^2 + \\ &+0.5\times q_{surcharge} \times K_a \times h_{excavation} \times (0.5\times h_{excavation}+ h_{penetration}) + \\ &+ (1/6) \times\gamma_w\times (h_w+ h_{penetration})^2 \qquad \text{(Eq 6.11)}\end{aligned}$$

The resistant moment is calculated as:

$$\sum M_R= (1/6)\, \gamma'_{soil}\times K_p \times h_{penetration}{}^2 +0.5\times \gamma_{wall} \times B \times (h_{excavation}+ h_{penetration}) \qquad \text{(Eq 6.12)}$$

The factor of safety is calculated as:

$FOS = \sum M_R / \sum M_O$

For permanent forces $FOS_{sure} \geq 2$, for seismic and wind forces: $FOS_{sure} \geq 1.5$, and for temporary wall: $FOS_{sure} \geq 1.25$ were recommended by BS8002 [83].

There are alternatives to increase FOS against overturning during design. Increasing the width of wall, increasing the penetration of wall, and use of strut supporting system or anchoring, are several alternatives to grow stability against overturning. In case of limited wall penetration, anchor pre-stressing prepares condition to prevent overturning failure (Sabatini et al.) [83]. EC7 recommended the partial factors for design approach based on limit state for overturning analyzing which indicated in table 6.6 [29, 30, 50, 71, 87]. Although partial factors are important for design, but it seems in construction and risk management use of FOS is simpler than converting partial factors to a one FOS.

Table 6.6: The partial factors for design approach in overturning analysis [29, 30,71]

Partial load factors:	
Permanent action - unfavorable	1.10
Permanent action - favorable	0.9
Variable action – unfavorable	1.5
Variable action – favorable	0
Partial materials factors:	
Soil parameter : $\tan\phi'$	1.0
Soil parameter : c'(effective cohesion)	1.0
Soil parameter : s_u (undraind shear strength)	1.0
Soil parameter : unconfined strength	1.0
Soil parameter : weight, density	1.0
Partial resistance factors: Driven pile	1.0

6.4 Bearing capacity

The ultimate bearing capacity is the significance bearing stress due to shear failure which causes a sudden settlement of the retaining wall on soil and is given by Terzaghi formula (only for long strip shallow footings, vertical none-eccentric loading) [66,77, 88]:

$$qu = c \times N_c + \gamma_{soil} \times D_f \times N_q + 0.5 \times \gamma_{soil} \times B \times N_\gamma \qquad \text{(Eq 6.13)}$$

where, $c = c' + (u_a - u_w) \tan\phi_b$, and N_c , N_q , N_γ = bearing capacity factors for footing surface (depend on ϕ and the base roughness), and D_f is depth of founding.

In cases where B < 3 meters (deep excavation retaining wall), $0.5 \times \gamma soil \times B \times N\gamma$ could be neglected with little error [13, 30, 63].

$$q_u = c \times N_c + \gamma_{soil} \times D_f \times N_q \qquad \text{(Eq 6.14)}$$

where, $N_q = e^{\pi \times \tan\phi'} \times \tan^2 [45^o + \phi'/2]$ and $N_c = (N_q - 1) / \tan\phi'$

The stress at founding level (under wall penetration) is reassured by the removal of the weight of soil [77, 88].

$$q_o = \gamma \times d$$

where, d is the founding depth and γ is the unit weight of the soil removed.

The net bearing pressure (q_n) is the increase in allowable stress on the soil [77, 88, 66, 67].

$$q_n = q_{allow} - q_o$$

The allowable bearing capacity (qallow) is normally calculated from the ultimate bearing capacity using a factor of safety as:

$q_{allow} = [(q_u - q_o) / FOS] + q_o$

where FOS is factor of safety which reversely can be estimated as:

$FOS = (q_u - q_o) / (q_{allow} - q_o)$

For rectangular foundations such as diaphragm retaining wall and for circular foundations such as bored-pile wall with drilling in situ, shape factors and depth factors are implied in bearing capacity formulae:.

$q_u = c \times N_c \times [\text{shap f(c)}] \times [\text{depth f(c)}] + \gamma_{soil} \times D_f \times N_q \times [\text{shap f(q)}] \times [\text{depth f(q)}]$

Shape factors are suggested by Brinch Hansen and Vesic based on ϕ'. However, with sufficiently accuracy the quantities of table 6.7 for shape factor and table 6.8 for depth factor are usually used (D is vertical distance between surface level and base level, and B is width of wall on base).

FOS for permanent foundation has to be between 2.5 and 3.0 but for temporary structures should be larger than 1.5 in short terms. Allowable bearing stress for soils without settlement considering is given by BS 8004 for use in preliminary design.

For a typical unsaturated underground soil the ultimate bearing pressures vs. matric suction on the basis of a constant ϕ' (20^o), ϕ^b (15^o), c'(5 kPa), γ (18 kN/m^2), and 0.5 meter depth from surface with 0.5 meter footing width is shown in figure 6.5 [83, 84].

As a vertical load P (include weight of wall) is exist in retaining wall, the maximum and minimum bearing stress in area under footing of retaining wall could be calculated as:

$q_{max} = (P/B) + (6\times e)/B$

or:

$q_{min} = (P/B) - (6\times e)/B$

where B is wide of wall footing and e is eccentricity of load on wall wide. For $e \geq B/6$ then $q_{min} = 0$, and

$q_{max} = (2\times P) / [(1.5\times B) - e]$

It is requiring that q_{max} is less than allowable bearing capacity:

$q_{max} \leq q_{allow}$

There are presumed allowable bearing stress for kind of grounds by BS8002 [83] for example: more than 600 kN/m² for dense gravel or dense sand and gravel, less than 200 to 600 kN/m² for medium gravel or medium sand and gravel, less than 200 kN/m² for loose gravel or loose sand and gravel, 100 to 300 kN/m² for medium sand, less than 100 kN/m² for loose sand, 150 to 300 kN/m² for stiff clays, and less than 75 kN/m² for soft clays and silts.

Table 6.7: Shape factors for soil ultimate bearing capacity

Shape	**shap f(c)**	**shap f(q)**
Square	1.3	1.2
Circle	1.3	1.2
Rectangle (B=breath, L=length)	$1+0.2\times(B/L)$	$1+0.2\times(B/L)$

Table 6.8: Depth factors for soil ultimate bearing capacity

Depth condition	**depth f(c)**	**depth f(q)**
$D > B$	$1+0.4\times\arctan(D/B)$	$1+2\times\tan(\phi'(\sin\phi'))^2\times\arctan(B/D)$
$D \leq B$	$1+0.4\times(D/B)$	$1+2\times\tan(\phi'(1-\sin\phi')^2)\times\arctan(B/D)$

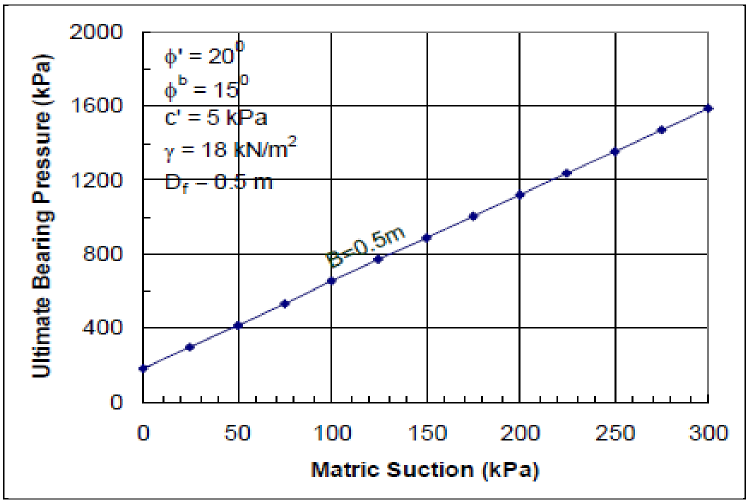

Figure 6.5: Typical unsaturated underground soil the ultimate bearing pressures vs. matric suction on the basis of a constant and 0.5 meter depth from surface with 0.5 meter footing width [83, 84]

6.5 Basel heaves

After construction of retaining wall and any excavation stages there is possibility that due to weight of retained soil or excessive vertical pressures on retained soil, heave failure occurred on excavated lot. Based on the moment equilibrium method and details of figure 6.6 for alone wall, and figure 6.7 for final level of propped wall, the factor of safety (FOS) could be calculated [4].

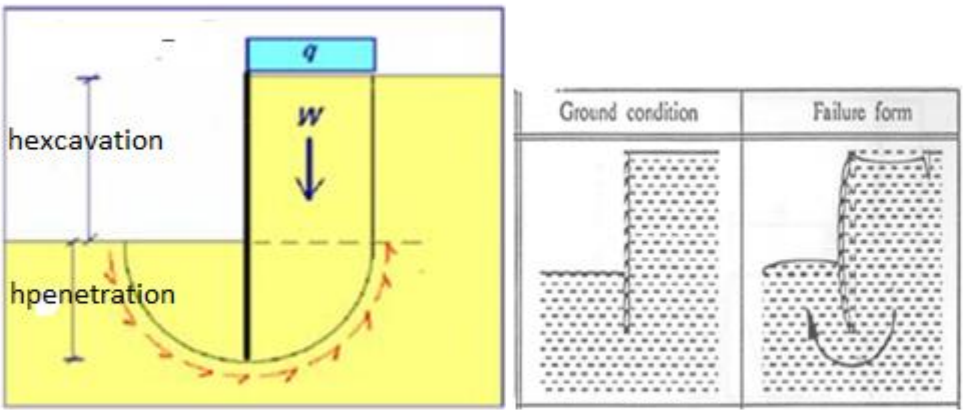

Figure 6.6: Schematic Basel heaves (retaining wall without probe) [4]

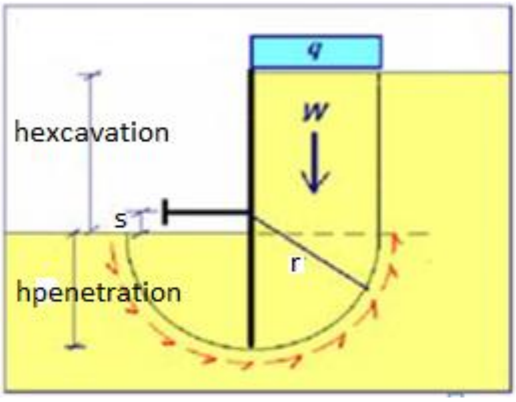

Figure 6.7: Schematic Basel heaves (retaining wall with probe) [4]

FOS = ($s_u \times \pi \times h_{penetration}^2$) / [(W + q× $h_{penetration}$) × $h_{penetration} \times 0.5$] (without prope)

FOS = [su× (π×r2 - 2×s×r)] / [(W + q× $h_{penetration}$) × r ×0.5] (one level prope)

where q is surcharge load, s_u is undrained shear strength of soil,

W = total weight of soil = $\gamma_{soil} \times h_{excavation} \times h_{penetration}$ (no probe)

W = total weight of soil = $\gamma_{soil} \times h_{excavation} \times r$ (with probe)

$r = h_{penetration} + s$

$FOS_{sure} \geq 1.2$

Note: the vertical shear resistance along the retained ground shallower than the excavation is ignored [4].

A limited range for relationship between FOS in opposition to Basel heave versus normalized maximum lateral wall deflection was proposed by Mana and Clough that with some observations of case histories at stages 3-7 of excavation is shown in figure 6.8 [6].

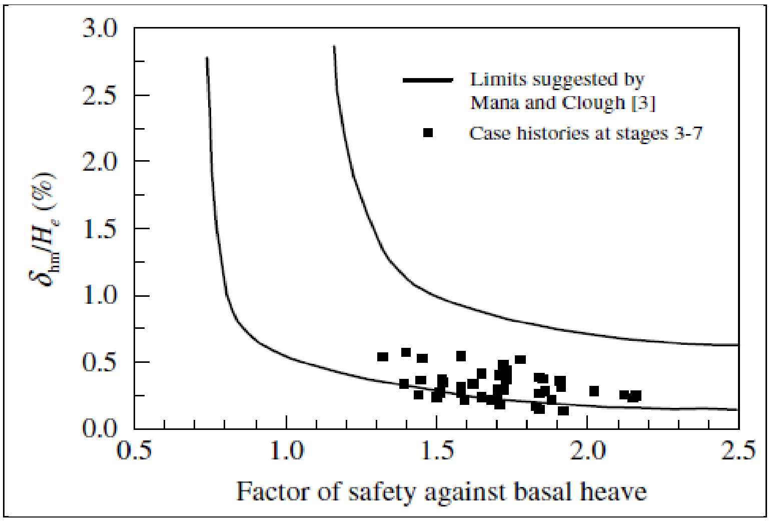

Figure 6.8: Relationship between factor of safety in opposition to Basal heave and normalized maximum lateral wall deflection [6]

6.6 Bottom heave due to unloading

Bottom heave usually occurs in soft and weak clay soils rather than in stiff clays and cohesionless soils. It is in fact a bearing capacity problem due to unloading, and following expressions are proposed [3]:

$FOS = s_u N_c / (\gamma_{soil} d_e + q)$ for $d_e/X>1$, N_c : 6 -7

$FOS = s_u N_c / (d_e (\gamma_{soil} - s_u /Y))$ for $d_e/X<1$, N_c : 7- 8

where s_u is undrained shear strength, N_c is bearing capacity factor, d_e is depth of excavation, q is surcharge, X is width of excavation, and Y is distance from bottom of excavation to stiff layer.

If there is no stiff layer, $Y = 0.7\times X$ is used. Factor of safety (FOS) is recommended as 2 in this case. For FOS between 1.5 and 2.0 heaving displacements are observed [3].

6.7 Heaven failure due to artesian pressure

Hidden artesian pressures under relatively impermeable soils may cause blow out of base resulting in submergence of excavation pit [4]. Based on the moment equilibrium method and details of figure 6.9, the factor of safety (FOS) could be calculated as:

$$FOS = \frac{w}{u}$$

where w = overburden pressure (γ_{soil} ×h) and γ_{soil} is soil bulk unit weight,

u = pore water pressure

$FOS_{sure} \geq 1.2$

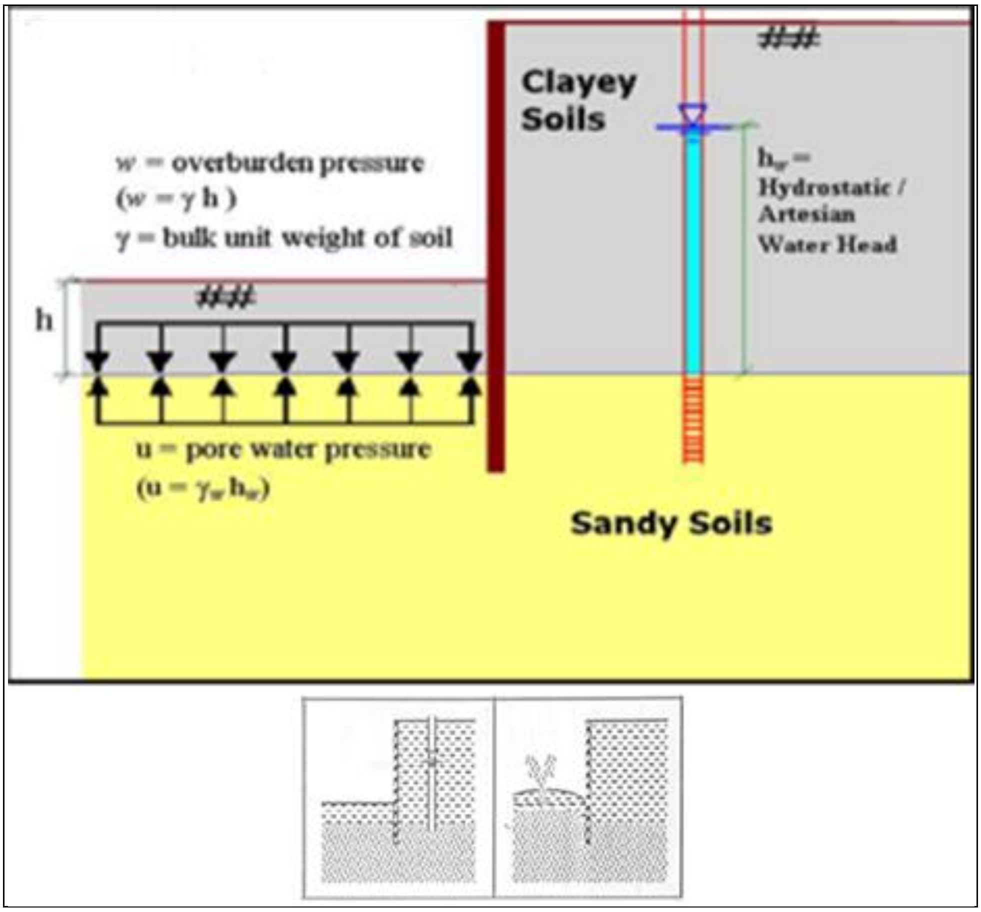

Figure 6.9 Heaven due to artesian pressure [4]

6.8 Upheaval failure

Hidden water pressures under relatively impermeable soils may cause blow out of base resulting in aquifer or an underground water resource. Based on details of figure 6.10, the factor of safety is recommended FOS ≥ 1.5 [86] which is calculated by the formula:

$FOS = (\gamma_1 \times h_1 + \gamma_2 \times h_2) / P_w$

where $P_w = H_w \gamma_w$

For more layers: $FOS = (\sum \gamma_{ti} h_i) / (H_w \gamma_w)$

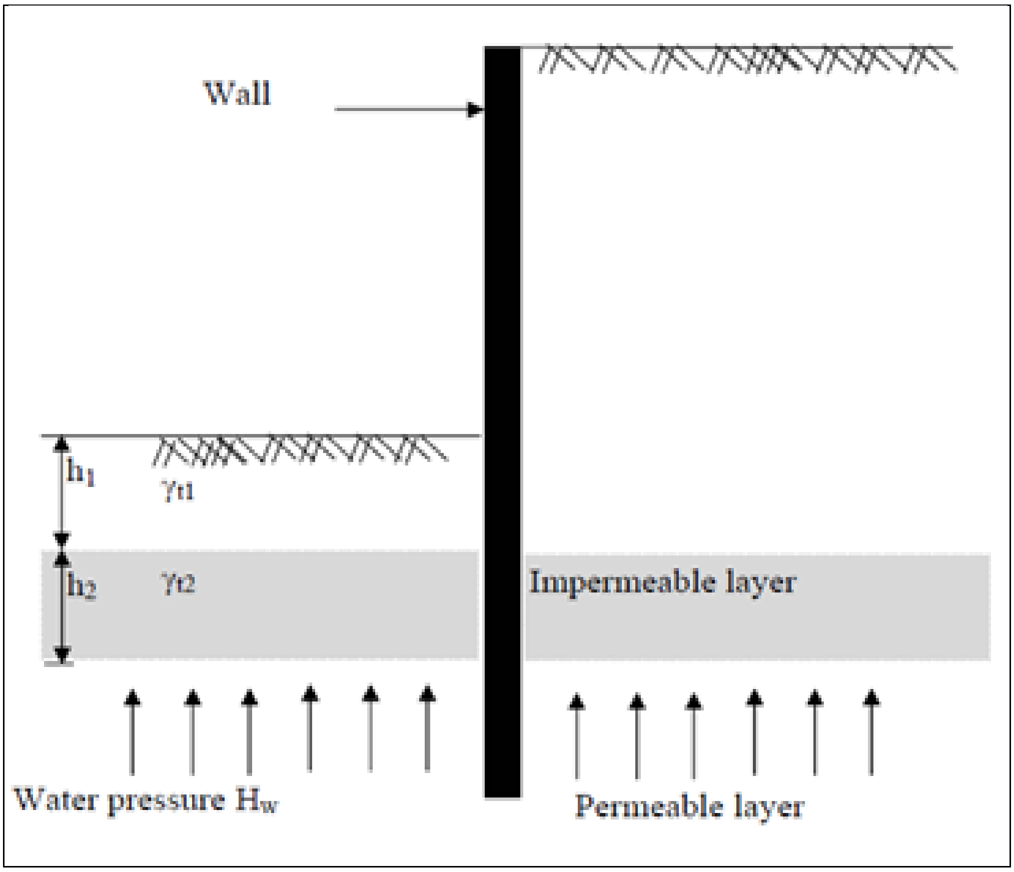

Figure 6.10 Diagram for upheaval failure [86]

6.9 Hydraulic failure (Piping)

The piping is initiated by a flow caused soil particle transport on the free unloaded downstream soil surface [59]. Figure 6.11 shows the cases and stages to reach the piping failure. Based on the moment equilibrium method, the factor of safety (FOS) is estimated for two cases which are shown in figure 6.12 [4] .This type of failure may occur in dense sand or stiff cohesive soil.

In Terzaghi method for initial case, the factor of safety is estimated by:

$FOS = w/u = (2\times\gamma' \times L_d) / (\gamma_w \times h_w)$

Where for temporary works $FOS \geq 1.2$ and for permanent works $FOS \geq 1.5$ is recommended [86].

In critical case, the factor of safety is estimated by:

$FOS = i_c / i = [(G_s - 1) L] / [(1+e) h_w] = (\gamma' \times L) / (\gamma_w \times h_w)$

Where G_s is specific gravity of soil particle, e is void ratio, γ' is submerged unit weight of soil, and γ_w is unit weight of water.

$FOS_{sure} \geq 2$ is recommended [86].

It is recommend that piping failure should be avoided by adopting prescriptive measures such as: the use of filters, preventing seepage, and increasing the seepage path length [86]. The EUROCODE 7 partial factors for hydraulic failure analysis are illustrated in table 6.9.

Table 6.9: The partial factors for hydraulic failure analysis (EUROCODE 7)

Actions and corresponding soil parameters	**Combination**
Partial load factors:	
Permanent action - unfavorable	1.35
Permanent action - favorable	0.9
Variable action – unfavorable	1.5
Variable action – favorable	0
Partial materials factors:	
Soil parameter : $\tan\phi'$	1.0
Soil parameter : c'(effective cohesion)	1.0
Soil parameter : weight, density	1.0

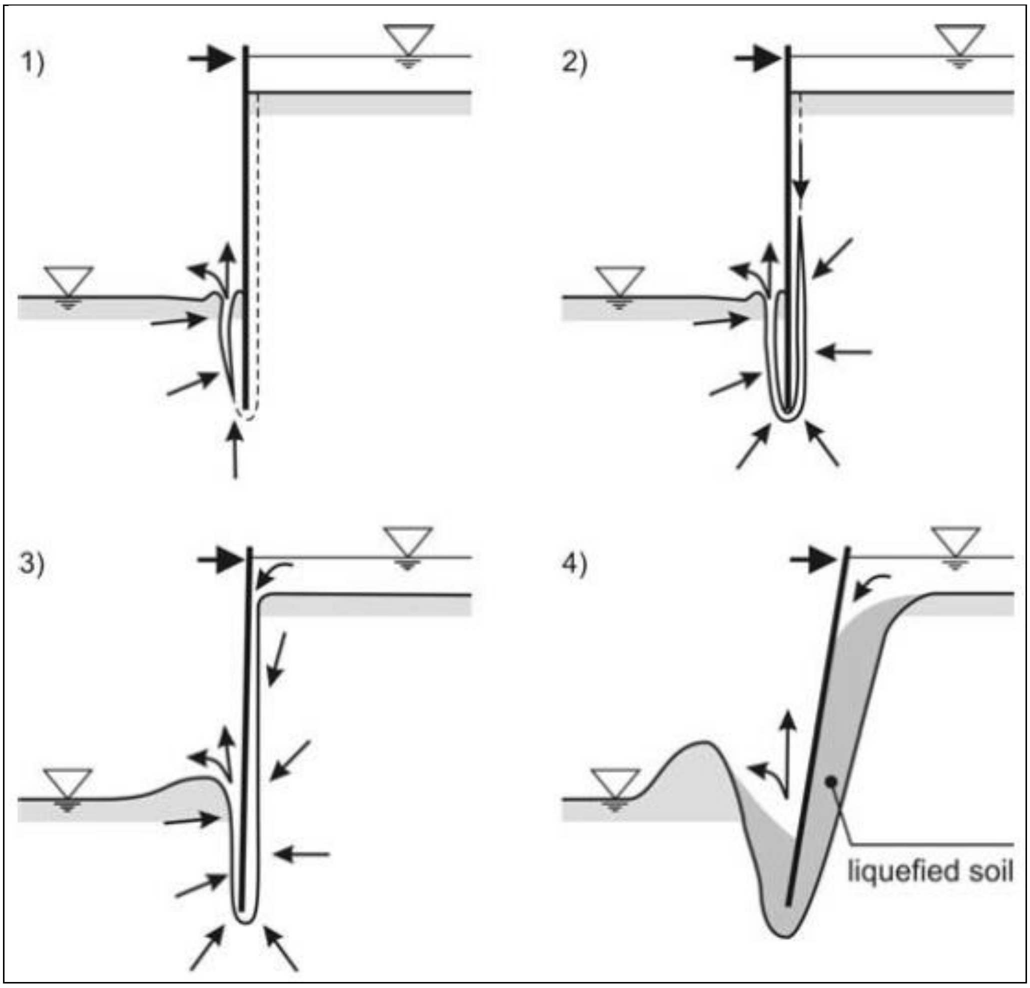

Figure 6.11: Stages to reach piping instability [59]

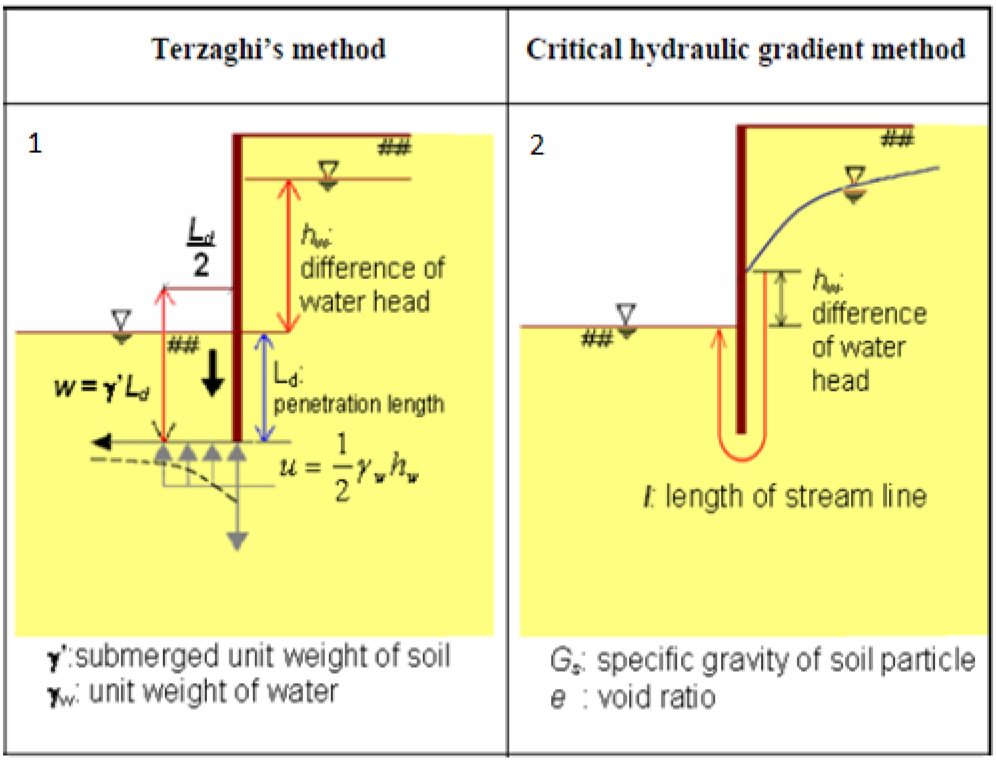

Figure 6.12: Diagrams in two cases for hydraulic failure [4]

6.10 Sand boiling failure

Piping in cohesionless soils, if not prevented, causes boiling near base inside and loss of passive resistance, and water and soil transferred inside through large openings leads to failure of wall. Based on the moment equilibrium method and figure 6.13, the factor of safety (FOS) is calculated as [86].

$$I_{critical} = \frac{\gamma}{\gamma_w}$$

$$I_{averag} = \Delta H_w / [h_{excavation} - h_1 + h_2 + 2\times (h_{penetration} - h_2)] \qquad \text{(Eq 6.17)}$$

$$FOS_{sour} = I_{critical} / I_{averag} \geq 1.5$$

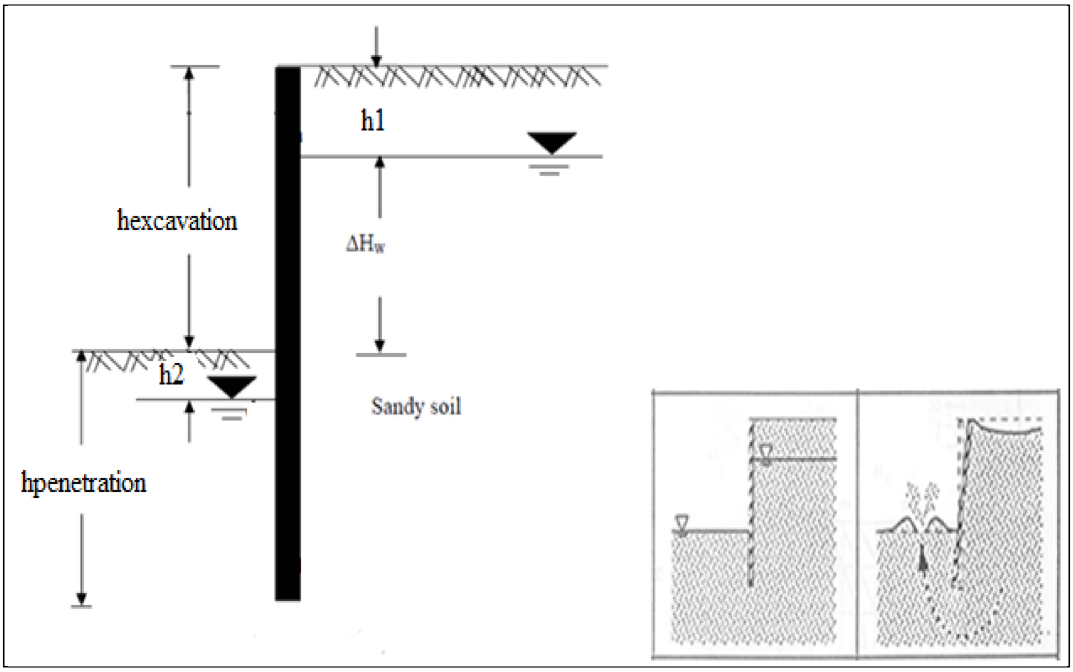

Figure 6.13 Diagram for boiling failure [86]

6.11 Liquefaction

Cyclic shear stress ratio (CSR) is proposed to estimate the liquefaction potential of soils by Seed et al [89] as:

$$CSR = 0.65\times (a_{max}/g) \times (\sigma_v/\sigma'_v) \times r_d \qquad \text{(Eq 6.18)}$$

where σ_v is the total vertical stress at depth h; σ'_v is the effective vertical stress at depth h; a_{max} is the peak horizontal acceleration of ground surface or each layer depend on studied case; g is the acceleration of gravity; and r_d is the stress reduction factor which depends on depth from surface.

r_d = 1- 0.00765×h for ($h \leq 9.15$ meters)

r_d = 1.174 - 0.0267×h for ($9.15 < h \leq 23$ meters)

The ratio of (σ_v/σ'_v) is $\gamma_{soil}\times h$ where γ_{soil} is unit weight of soil and h is depth below ground surface.

The liquefaction resistance of soil against earthquake with 7.5 magnitudes is proposed by Seed and Idriss [90] as cyclic resistance ratio (CRR):

$CRR_{7.5} = [1/ (34\text{-}N)] + [N/136] + [50/[(10\times N) +45]^2] - [1/200]$

where N is the number of blows/ft (N_{60}) in standard penetration test (SPT) of soil.

The CRR estimates the capacity of soil to resist liquefaction, which can be obtained from empirical correlations of the SPT. The liquefaction resistance of soil against earthquake with 6.5 magnitudes is evaluated by Youd et al [91] as:

$CRR_{6.5} = CRR_{7.5} \times MSF$

where $MSF= 10^{2.24}/ M^{2.56}$, and M is earthquake magnitude.

Factor of safety is estimated by:

$FOS = CRR_{6.5}/ CSR$

FOS >1 non-liquefiable, and FOS ≤1 liquefiable

The severity of foundation damage caused by soil liquefaction cannot be accessed directly by the FOS, mostly in cases that depend on the severity of liquefaction. For the evaluation of the liquefaction hazard, Iwasaki et al. proposed the liquefaction potential index (LPI). The LPI evaluates liquefaction potential over the length of a boring. A weighting function gives higher values to the layers closest to the ground surface, and decreases linearly to zero at a depth of about 20 m [92, 93].

$LPI = \Sigma (F_i\times W_i\times t_i)$ (Eq 6.19)

Where t_i is the thickness of the each layer, F_i denotes the liquefaction severity for layer i which is 1 for liquefaction or zero for otherwise, and W_i is weighting function gives higher values to the layers closest to the ground surface, and decreases linearly to zero at a depth of 20 m so that $W = 10 - 0.5\times z$ for $z \leq 20$ m.

Liquefaction risk assessment is categorized by the LPI that are shown in table 6.10. All the categories divide the LPI and severity of liquefaction in four sections. In this way the severity jumps from low to high without any moderate section severity. The liquefaction of clayey sands is indicated in table 6.11 [94] which indicated the relation between plasticity index (PI) of soil reversely with liquefaction severity and also the use of clays in sand as a percent against liquefaction.

The ‘pseudo velocity’ is a parameter of earthquake motion on surface that can be a reliable measure of earthquake severity in the field [95] which presents the earthquake nature. The‘pseudo velocity’ varies along the depth of underground which the variation is shown in figure 6.14 [95]. The V_{ref} is assumed the Pseudo velocity at 15 m below the ground surface [95]. The liquefaction risk is low when V_{ref} is relatively low [95]. The interested points such as maximum pore pressure and depth of liquefaction are surveyed [95]. Figure 6.15 shows the variation of LPI vs. maximum pore pressure ratio [95]. The variation of depth of liquefied region vs. depth of soil deposit (H_L/H) for different LPI is shown in figure 6.16 [95]. Figure 6.17 shows the relationship between V_{ref} vs. D_r% in presence of line LPI=5 and liquefaction potential.

EC8 [96] states that the risk of liquefaction may be neglected when $(a_{max}/g) < 0.15$ and, at least, one of the following conditions is satisfied:

●The sands have a clay content greater than 20% with a plasticity index PI >10

●The sands have a silt content greater than 35% and an SPT blow count, normalized for overburden effects and the energy ratio, of N >20

●The sands are clean, with an SPT blow count, normalized for overburden effects and the energy ratio, of N >30.

Table 6.10: Liquefaction risk assessment categories [92, 97, 98, 99]

LPI	**Iwasaki et al** [92]	**Chung and David**[97]	**Iwasaki et al** [98]	**Choong-Ki et al** [99]
0	Not likely	None	Very low	None
0 < LPI≤5	Minor	Little to none	Low	Low
5 < LPI≤15	-	Moderate	High	High
15 < LPI≤100	Severe	Severe	Very high	Extreme

Table 6.11: Liquefaction of clayey sands [94]

clayey sands characters	**Non-plastic sand**	**Low plasticity clayey sand**	**Medium plasticity clayey sand**	**High plasticity clayey sand**
Plasticity Index, PI	0	≤ 4	5-14	≥15
Liquefaction potential	liquefaction	Rapid liquefaction	Liquefaction; but liquefaction resistance increased	No liquefaction

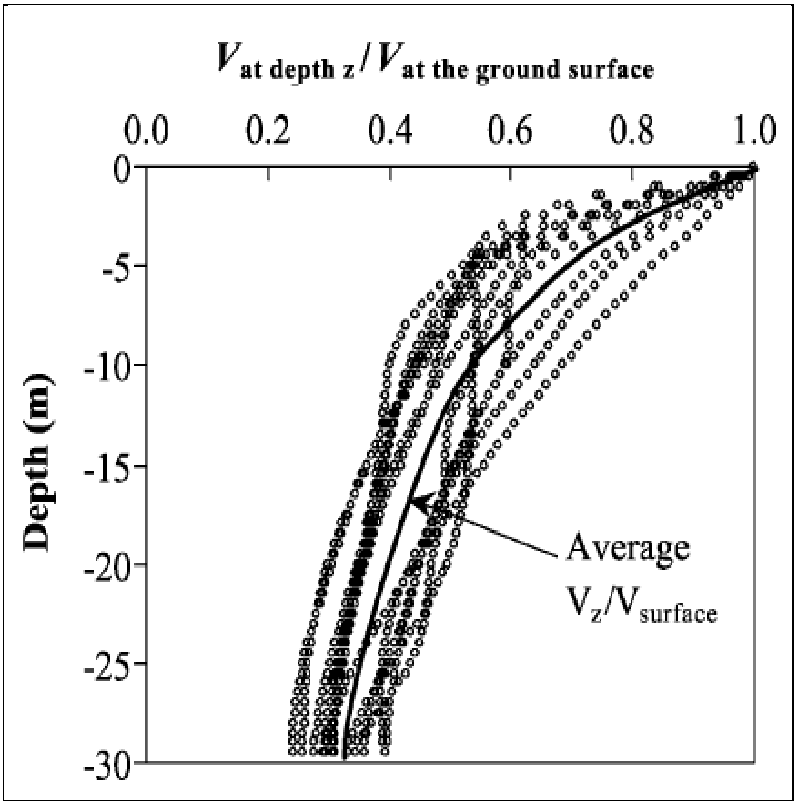

Figure 6.14: Variation of The'pseudo velocity' in depth for different important earthquakes [95]

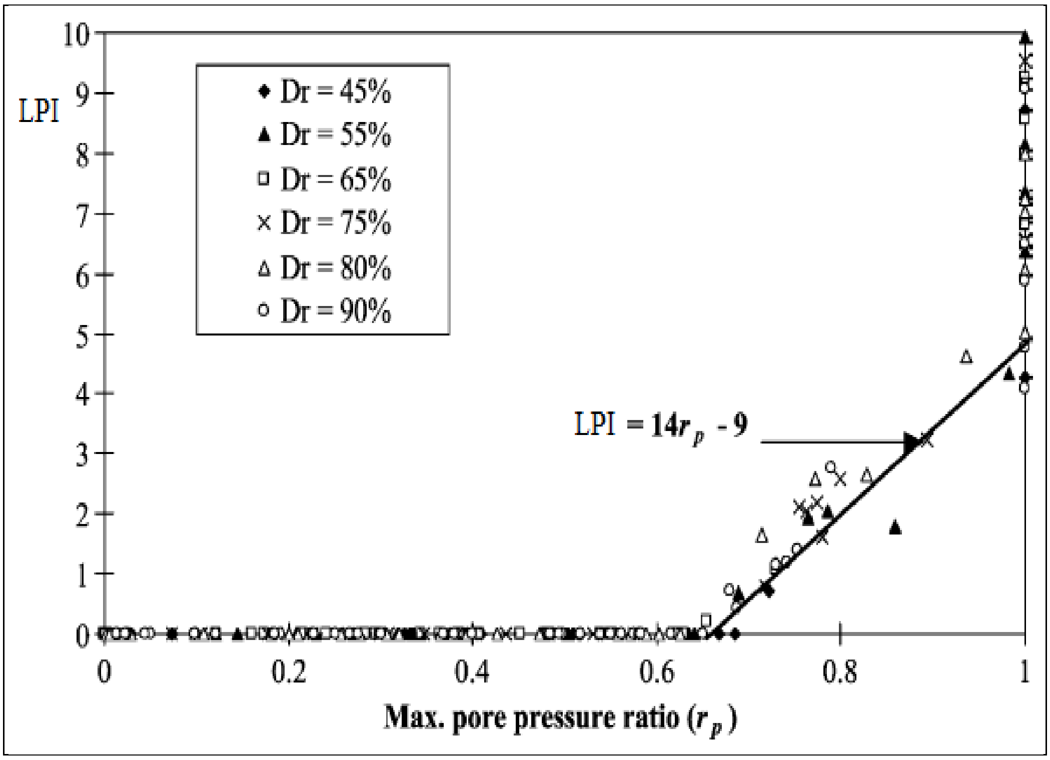

Figure 6.15: Variation of LPI vs. maximum pore pressure ratio in presence of D_r% [95]

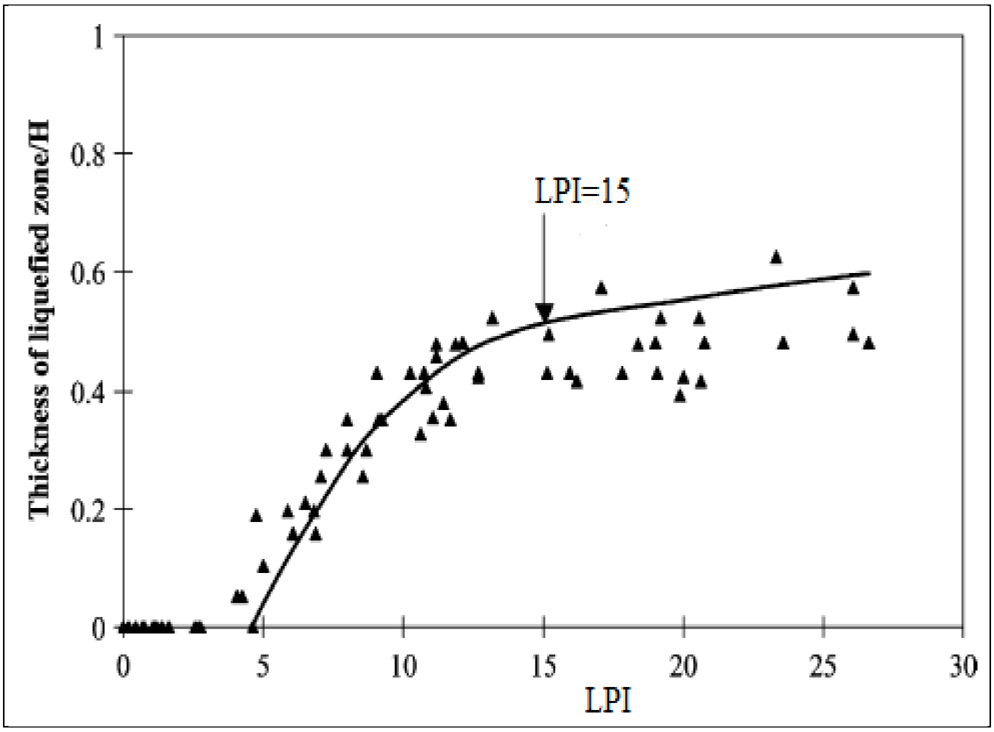

Figure 6.16: The variation of depth of liquefied region/depth of soil deposit (H_L/H) for different LPI [95]

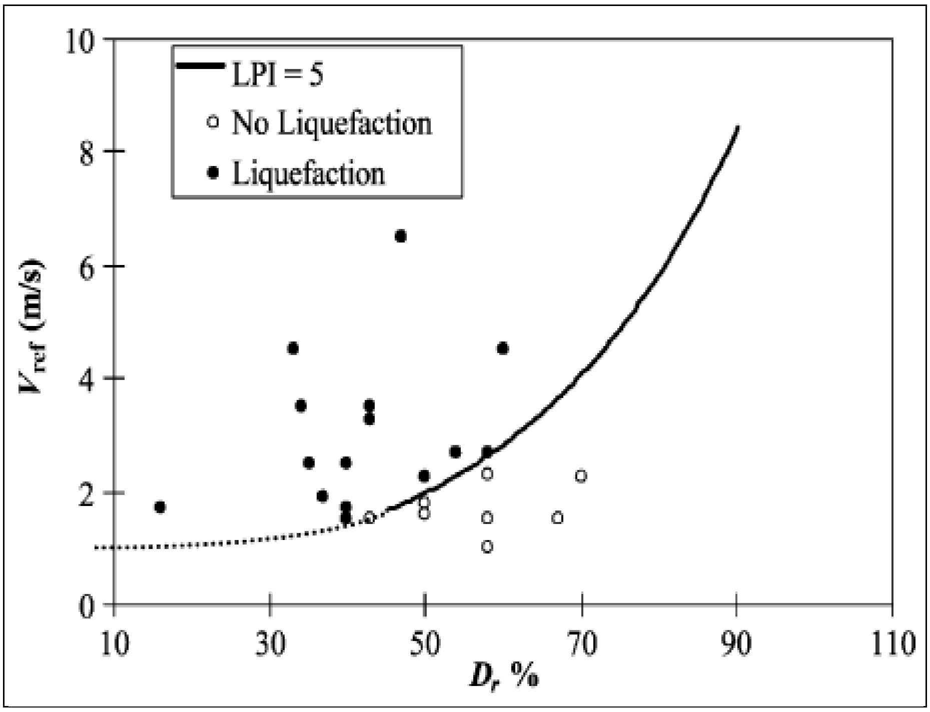

Figure 6.17: The relationship between V_{ref} vs. D_r% in presence of line LPI=5 and liquefaction potential [95]

6.12 Anchored wall failure due vertical load

Ground anchors are commonly installed at angles of 10–40 degree below the horizontal. Anchors should be installed as close to horizontal (15^o) as possible to minimize vertical loads resulting from anchor to wall. Accumulated vertical components of anchor forces could origin for bearing capacity failure of base in soft soils. Vertical displacement of wall impairs the anchors [4]. The cycle iterates and finally vertical failure of embedded wall is occurred that schematically is shown in Figure 6.18. For short term and less critical structure FOS ≥ 1.2 and for long term or high risk to life structures FOS ≥ 1.4 was recommended [83].

Kamal Mohamed Hafez Ismail Ibrahim et al (2013) studied effect of earthquakes on one raw tie back (12.5 m cable, horizontal angle 15°, grouting length 4 m, pre-stressing load100 kN/m, spacing 2.3 m, at 3.5 meters depth) diaphragm wall with 9.5 m excavation depth,11 m length, 0.5 m thickness, in ground with layers: $_U$pper sand: 3.5m, γ=19.5,ϕ=34, Silty clay: 3.5 m, γ=20, c_u=12, ϕ=15, and Lower sand: 43 m, γ=20, ϕ=38 by PLAXIS software[100]. The normal force in diaphragm wall owing to different historical earthquakes is illustrated in Figure 6.19. About 285 kN/m in 10.5 meters of wall length is maximum normal force in one meter of wall.

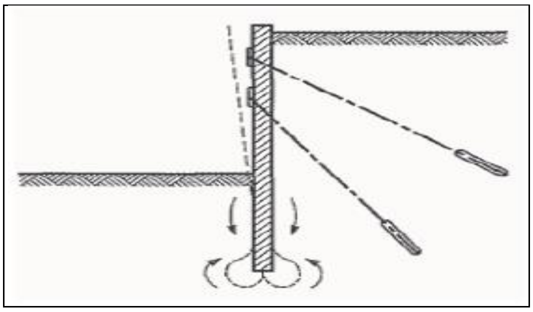

Figure 6.18: Vertical failure of two level anchored wall [61]

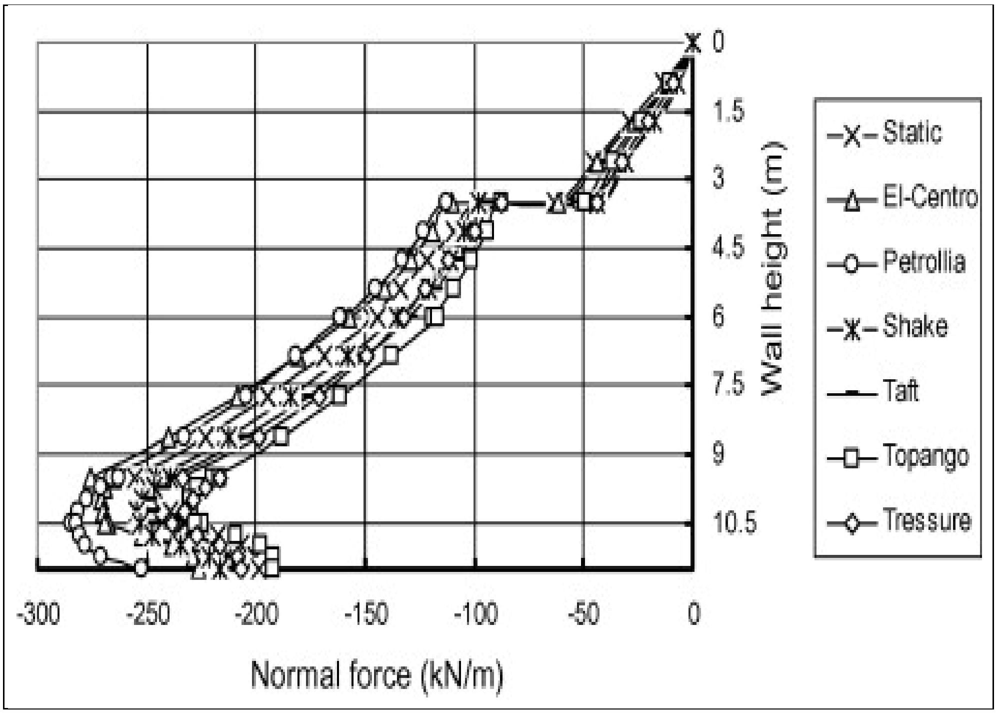

Figure 6.19: Distribution of normal force in diaphragm wall owing to different historical earthquakes [100]

6.13 Bending moment failure of anchored wall

The wall may fail reaching its structural capacity due to overloading. Some of the anchors may also get damaged (at the head or pull-out) after wall failure. Excessive deformations on very flexible walls sometimes cause problems at anchors even if wall retains its stability. A schematically bending moment failure of anchored wall is illustrated in Figure 6.20. Bending moment diagram due to different historical earthquakes [100] for one raw tie back diaphragm wall (Kamal Mohamed Hafez Ismail Ibrahim et al) is shown in figure 6.21. About 520 kNm/m is maximum bending moment in 8 meters of 11 meters length of wall while it is about 150 kNm/m in 3.5 meters (tie back location).

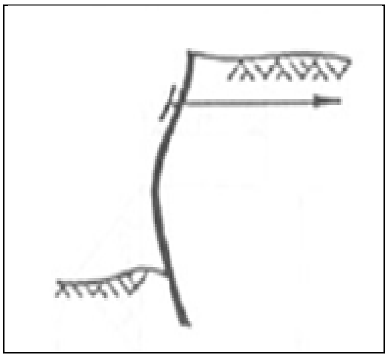

Figure 6.20: Bending moment failure of anchored wall [2]

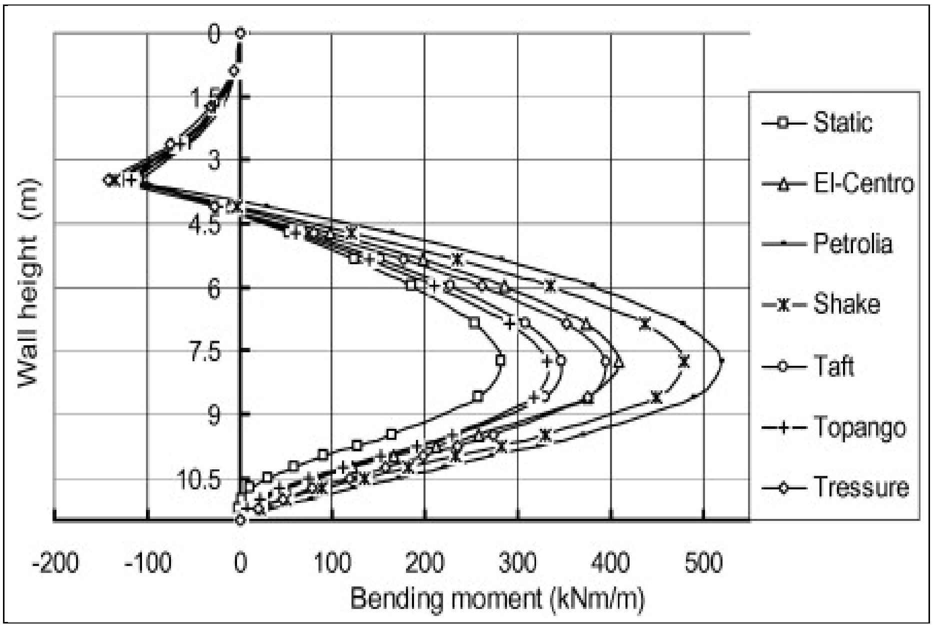

Figure 6.21: Bending moment in one raw tie back diaphragm wall owing to different historical earthquakes [100]

6.14 Cantilever wall failure by forward rotation

Due to expansion effect of expansive clays retained by wall, cantilever wall starts to forward rotation that may fail cantilever wall. The magnitude of passive force may influence on severity of failure. Cantilever wall failure by forward rotation is shown in figure 6.22 schematically. The exerted pressure by special expansive soils could reach even more than 3.8 to 36.7 kg/cm^2 which is more than three times to thirty six times of allowable bearing capacity of normal soils [101].

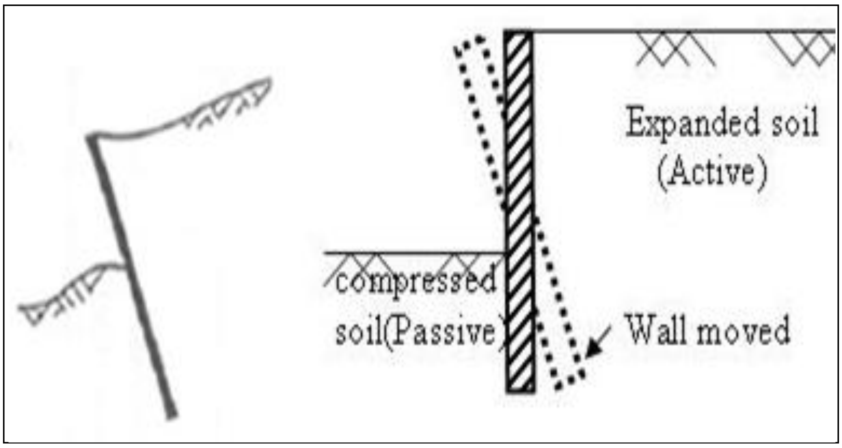

Figure 6.22: Cantilever wall failures by forward rotation [2]

6.15 Yield anchor failure

Insufficient numbers of anchors result in excessive anchor loads and anchors fail one by one leading to complete wall failure. Main possibilities may be collapse of ground/grout interface, tendon yielding or corrosion, and grout/tendon interface. Minimum recommended FOS for tendon, ground/grout interface, and grout/tendon interface in two cases of permanent and temporary is illustrated in table 6.12 [86]. Schematically Failure of yield anchor is illustrated in figure 6.23. The changes of Earthquake-induced force in anchor of tie back wall of section 6.13 (Kamal Mohamed Hafez Ismail Ibrahim et al) is shown in figures 6.24, 6.25, and 6.26. Figure 6.24 shows tie back force-time history which roses from 100 (pre-stressed) to 210 kN/m during excavation form level of -3.5 to level of -9.5 then due Petrollia earthquake drastically rocket to 350 kN/m during 3 second earthquake period. Figure 6.25 indicates that anchor excessive force rise from 260 kN/m to 287 kN/m when grouting stiffness changes from 50 to 3000 kN/m in a quasi parabolic curve. Tie back force at the end of earthquake reaches to 400 kN/m which is shown in figure 6.26.

Table 6.12: Minimum safety factors recommended for design of individual anchors [86]

Type of anchors	Ground/grout interface	Tendon	Grout/tendon or grout encapsulation interface
Temporary anchors	2.0	1.6	2.0
Permanent anchors	3.0	2.0	2.0

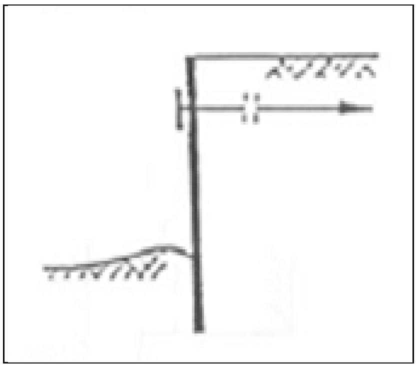

Figure 6.23: Failure of yield anchor [2]

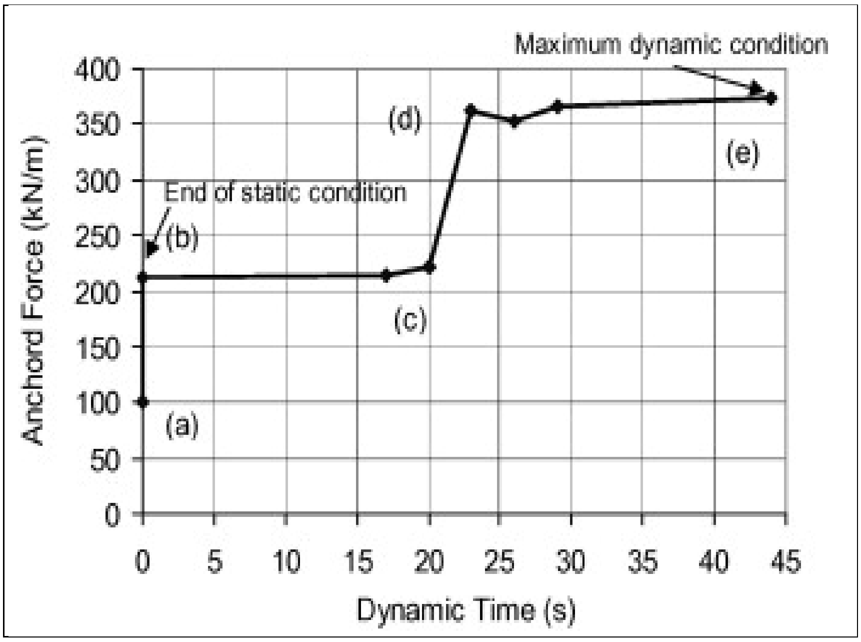

Figure 6.24: Tie back force-time history owing to Petrollia earthquake [100]

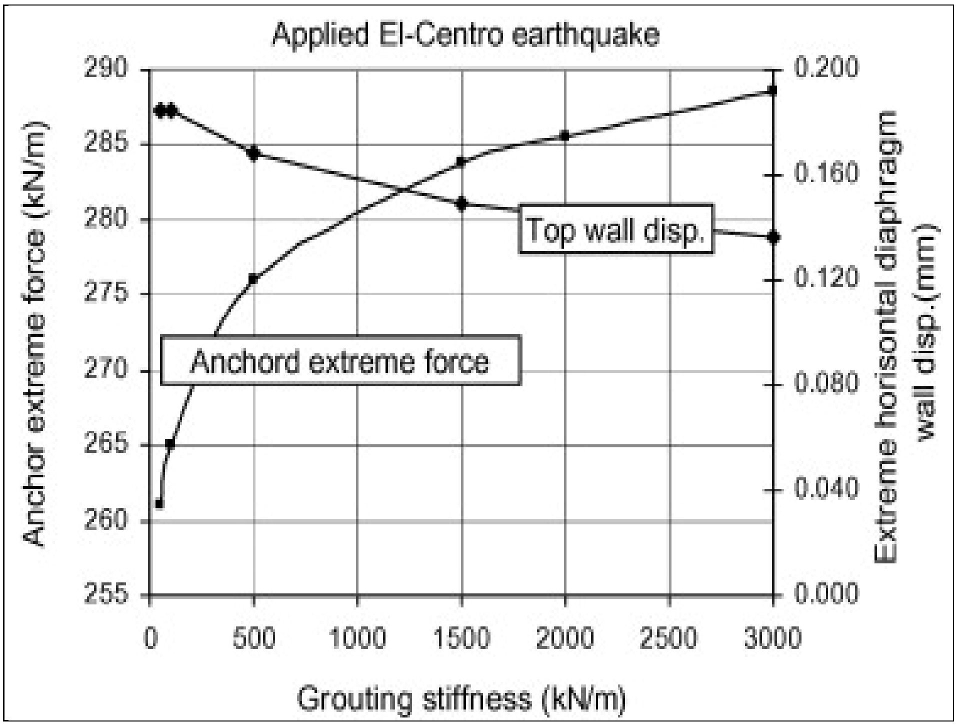

Figure 6.25: Effect of changing of grouting stiffness of tie back on intense tie back force and on diaphragm wall top dislocation [100]

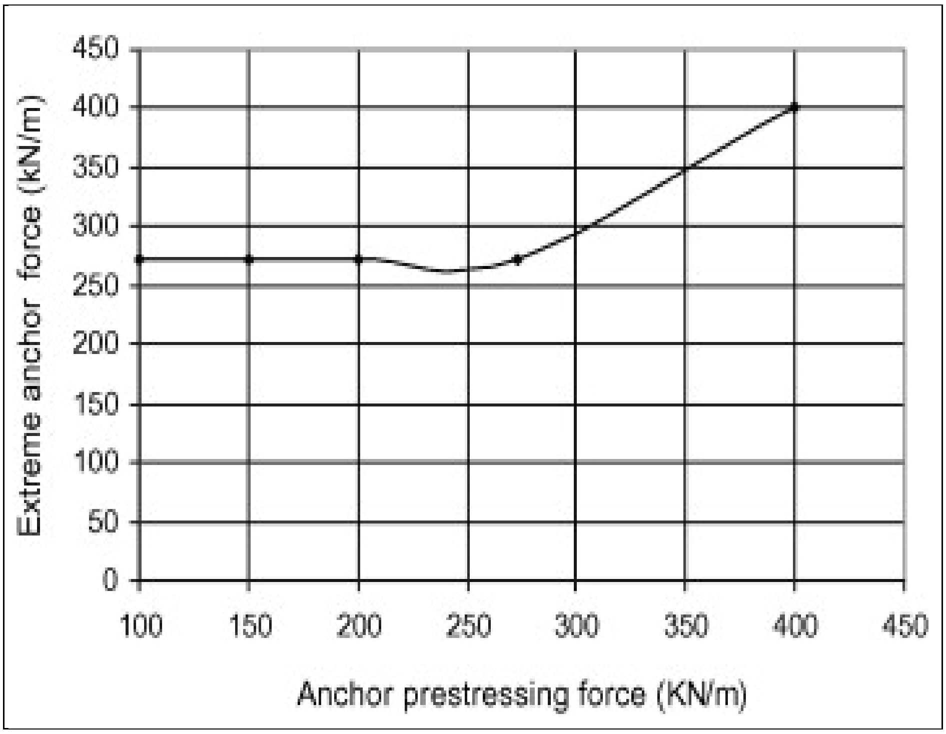

Figure 6.26: Effect of changing of tie back pre-stressed force on the intense final tie back force at the end of earthquake [100]

6.16 Failure of anchor supported wall by rotation about anchor

This failure is due to relatively weak wall and strong anchor. Failure of anchored wall by rotation about anchor is shown in figure 6.27 schematically.

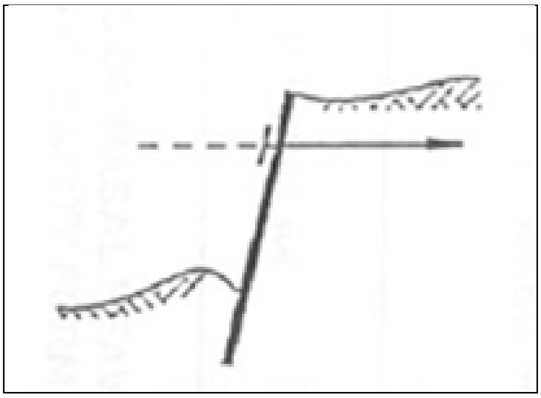

Figure 6.27: Failure of anchored wall by rotation about anchor [2]

6.17 Deep shear failure by rotation of soil mass

The wall penetration depth may be relatively short as it required in this case. Also it may be due to bearing capacity of supporting base sliding along the base. Schematically Failure by rotation of soil mass is illustrated in figure 6.28.

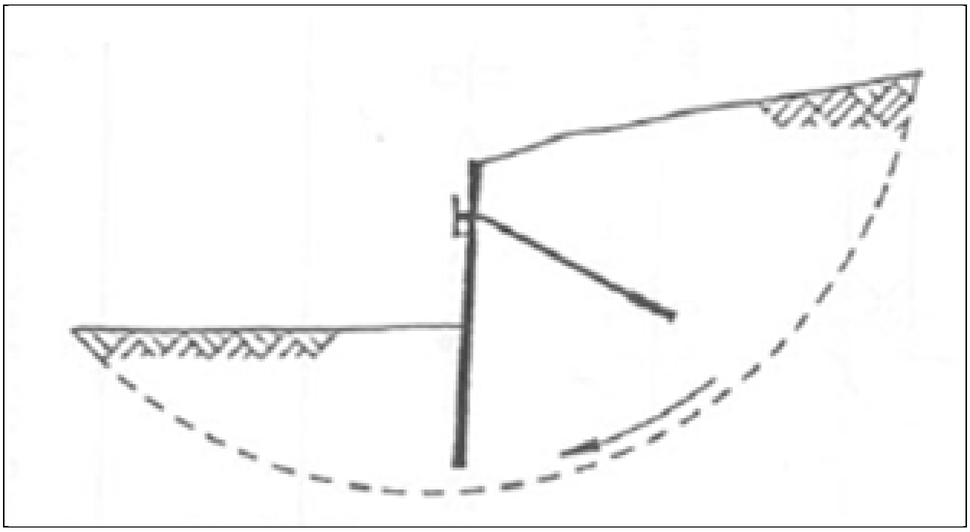

Figure 6.28: Failure by rotation of soil mass [2]

6.18 Overall instability failure due to low anchor length and short penetration depth of wall

In this case, both anchor length and wall penetration depth is short. Wall fails similar to overall instability failure but failure surface is located just behind anchors or at fixed length zone and below penetration depth. Figure 6.29 shows this position. For short term and less critical structure FOS ≥ 1.2, and for long term or high risk to life structures FOS ≥ 1.4 was recommended [86].

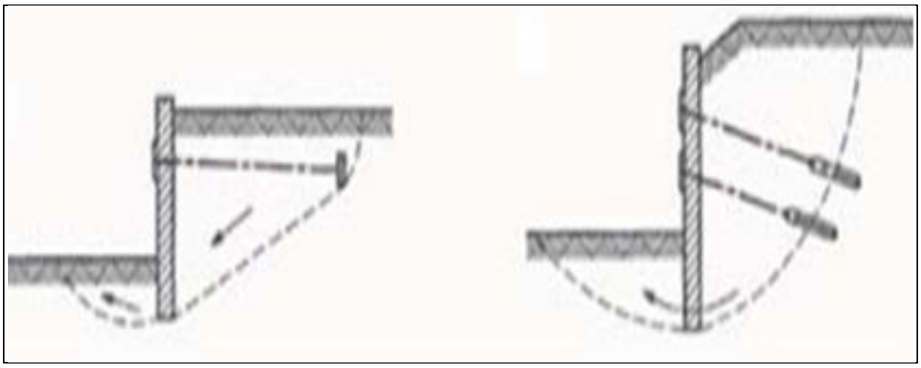

Figure 6.29: Overall instability failures due to low anchor length and short penetration depth [60, 61]

6.19 Overall instability failure due to low anchor length

In this case, the wall Penetration depth is sufficient but anchors are short and a large mass containing fixed length zone separates from soil mass behind and tilts forward. Figure 6.30 shows it schematically.

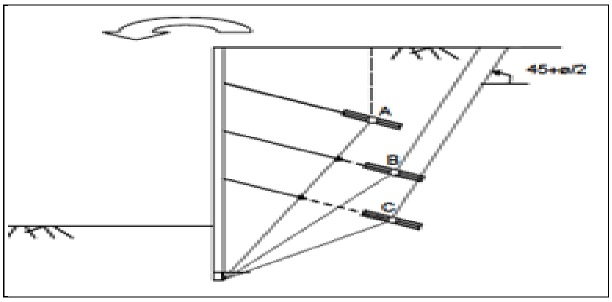

Figure 6.30: Overall instability failures due to low anchor length [67, 68]

6.20 Overall instability failure due to rotation of wall and anchors altogether

Slip surface is behind fixed length of anchors. Wall and anchors altogether rotate. This type of failure generally occurs in soft and weak soils and excavations in slopes. If instability detected at the time of project calculations longer anchor lengths and deeper embedment are designed. Figure 6.31 shows it schematically. For short term and less critical structure FOS ≥ 1.2 and for long term or high risk to life structures FOS ≥ 1.4 is recommended [86].

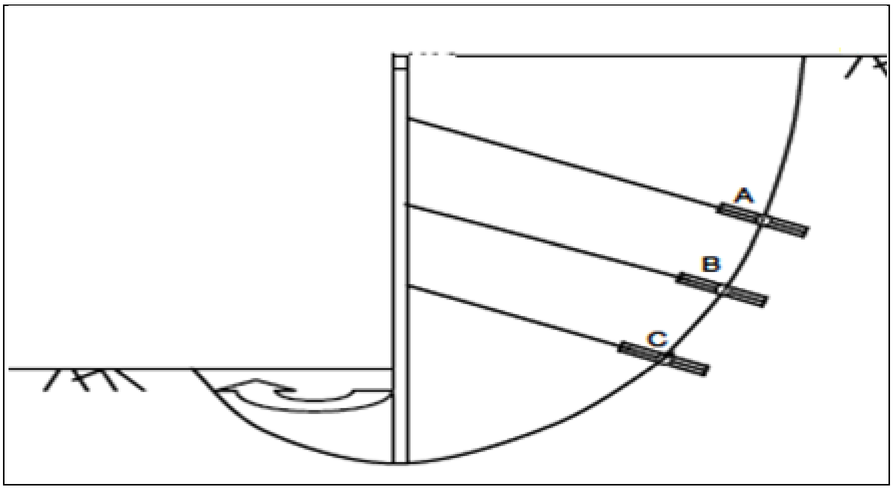

Figure 6.31: Overall instability failure due to rotation of wall and anchors altogether [60]

6.21 Passive zone failure

In this case the penetration of wall is sufficient but the passive zone is weak. Figure 6.32 shows this case schematically. Slip surfaces act on passive zone and passive force decreases so that wall rotates due to active force.

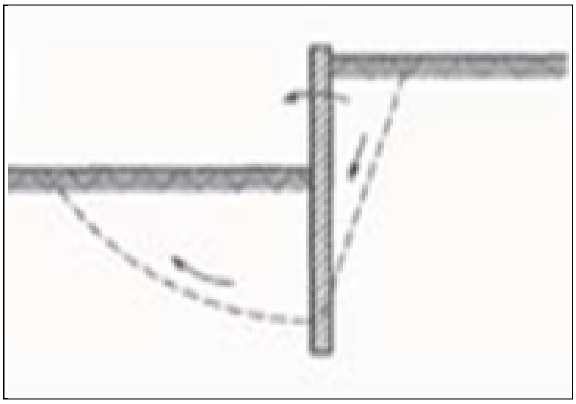
Figure 6.32: Passive zone failures [60, 61]

6.22 Failure of braced wall due to insufficient passive resistance

Failure of braced wall due to insufficient passive resistance, that trend to sliding is the important reason. It is shown in figure 6.33 schematically.

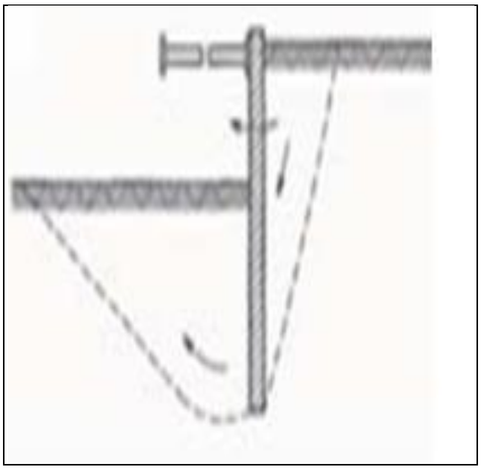

Figure 6.33: Failure of brace wall due to insufficient passive resistance [60, 61]

6.23 Failure of braced wall due to lack of bracing

In this case braced wall cannot carry excessive loading relative to lack of bracing (especially in top sections) which cannot give required resistant. Figure 6.34 shows Failure of brace wall due to excessive loading schematically. The excessive load may have different origins.

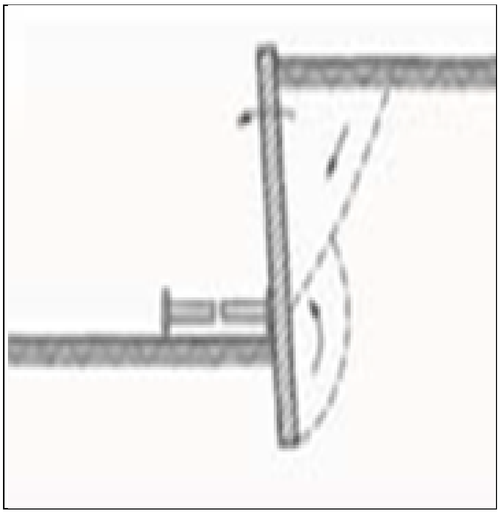

Figure 6.34: Failure of brace wall
due to excessive loading [60, 61]

6.24 Conclusion

Underground and/or support failures in twenty two (22) cases is reviewed which are risk register in deep excavation that each have situation for risk analyzing based on proposed formulas from factor of safety which is presented in chapter 5.

Bells method is developed in order to simplifying the sliding analysis for deep excavation.

Deep excavation more than between 20 to 23 meters depth in loose sand can mitigate liquefaction which may be improved by mixing clay or construction of about five to seven basements and multi floor building.

Summary of sure factor of safety for several different geotechnical or supports risks is shown in table 6.13 which obtains by overview.

Table 6.13 The FOS_{sure} for several different geotechnical and supports risks

Risk	**FOS_{sure}**	**FOS_{sure}**	**FOS_{sure}**
Sliding failure	Permanent support 1.75	Temporary support 1.25	
Overturning failure	Permanent support, 2	Temporary support, 1.25	Seismic, wind, 1.5
Bearing capacity	Permanent foundation, 2.5 to 3	Temporary foundation 1.5	
Basel heave failure	1.2		
Bottom heave failure	2		
Heaven due artesian pressure failure	1.2		
Upheaval failure	1.5		
Hydraulic-Piping (Terzaghi method)	Permanent works, 1.5	Temporary works, 1.2	
Hydraulic-Piping (Critical case)	2		
Sand boiling failure	1.5		
Anchored wall failure due vertical load	Short term and less critical structure, 1.2	Long term or high risk to life structures, 1.4	
Yield anchor failure, Ground/grout interface	Permanent anchors, 3	Temporary anchors, 2	
Yield anchor failure, Tendon	Permanent anchors, 2	Temporary anchors, 1.6	
Yield anchor failure, Grout/tendon or grout encapsulation interface	Permanent anchors, 2	Temporary anchors, 2	
Overall instability failure	Long term or high risk to life structures, 1.4	Short term and less critical structure, 1.2	

Chapter 7

GROUND MOVEMENT, SETTLEMENT, AND BUILDING DAMAGE

7.1 Introduction

Deep excavation may produce movement of retained ground, settlement under foundation of adjacent building, and the building repair or reconstruction damage. The important and interest point is distinction between deep excavation induced-settlement and ancient settlements or Greenfield induced-settlement as unrelated to deep excavation. This chapter collects related subjects and includes:

- Movement of ground, Settlement, Compressibility, and Ground stiffness
- Adjacent foundations movement due excessive retaining wall deflection
- Adjacent foundation movement limits
- Building damage classification due to excessive settlements in term of repairing state
- Ground cracks due to deep excavation

7.2Movement of ground, Settlement, Compressibility, and Ground stiffness

The movements of ground may be function of the origin of the soil, the structure of the soil particles, the bond between particles, water content, and historical loads. The movements of ground could be measured by inclinometers, gauges of timber and/or surveying instruments such as total station camera as displacements and rotations in three directions. Pore water pressure can be measured by piezometer in the field directly. Oedometer test is the laboratory test for ground samples to measure modulus E_{oed}. Triaxial compressibility and deformation testing is the laboratory test for ground samples to measure shear strength and internal friction angle. Ground movement splits to elastic (immediate), consolidation, and secondary consolidation (creep, and/or swell). Possibility of significant movements in different ground types could be presented in table 7.1 [88]. Schematically clay type ground movements in different situations as a function of time are shown in figure 7.1 [88].

Table 7.1: Possibility of significant movements in different types of ground [88]

Ground type	**Immediate movement**	**Consolidation movement**	**Creep movement**	**Swell movement**
Rock	Yes	No	No	Some
Gravel	Yes	No	No	No
Sand	Yes	No	No	No
Silt	Yes	Minor	No	Yes
Clay	Yes	Yes	Yes	Yes
Organic	Yes	Minor	Yes	Yes

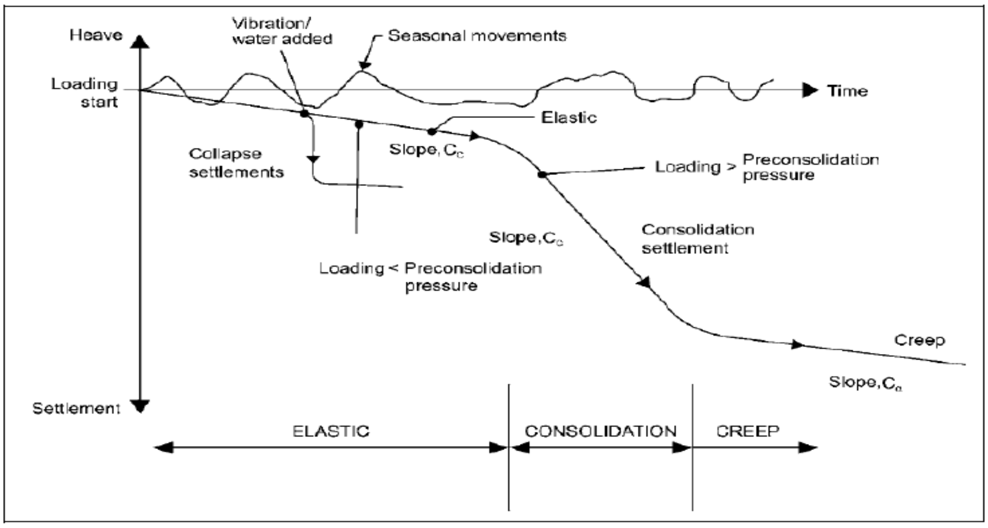

Figure 7.1: schematically clay type ground movements in different situations vs. time [88]

In elastic ground, elastic settlement of footing could occur that is:

$$S = \frac{q \times B \times (1-\upsilon^2) \times I_\rho}{E} \qquad \text{(Eq 7.1)}$$

Where

q = average pressure applied, B = width or diameter of footing,

υ = Poisson's ratio of the soil, E = Young's Modulus of the soil,

I_ρ = approximate influence coefficient for settlement

The influence coefficient (I_ρ) depends on footing shape, footing flexibility, distance to a rigid base, and footing embedment depth. The influence coefficient (I_ρ) is determined by figure 7.2 [66, 67]. Soil modulus should be determined by laboratory or field tests. Typical range of Poissons' ratio for different soils and range of soil Modulus in undrained state are indicated in tables 7.2 and 7.3 respectively. Figure 7.3 illustrates three soils, each one have a different value of compressibility and enclosed within rigid but frictionless boundaries. The settlement of the top of the soil due to $\Delta\sigma v$ is found by summing the contributions of each of the three soils.

$\rho = \Sigma$ (stress change × stressed length × compressibility)

or:

$\rho = \Delta\sigma v\,(z_A \times m_{vA} + z_B \times m_{vB} + z_c \times m_{vc})$

The compressibility of a soil is often measured by oedometer in a laboratory. The conventional method of plotting oedometer data using void ratios involves the use of a logarithmic scale for the stress. The plotted line for the first loading of the soil is often linear so the equation of the line can be expressed simply as follows:

$$e_f = e_0 - C_c \log_{10} (\sigma'_v + \Delta\sigma'_v) / \sigma'_v \qquad \text{(Eq 7.2)}$$

where the slope of the line, Cc, known as the compression index is an alternative measure of compressibility of the soil. The σ'_v is the initial value of the effective vertical stress. The settlement may be calculated by means of the compression index by use of the following expression in primary consolidation:

Settlement = layer thickness × strain = layer thickness × Δe / $(1 + e_0)$

or:

Settlement = layer thickness × $C_c \times \log_{10}[((\sigma_v' + \Delta\sigma'_v)/\sigma'_v)/(1 + e_0)]$

The value of the compression index (Cc) is normally defined as the slope of the virgin compression part of the e - log (σ'v) curve. Typical range of primary compression index (C_c) for different soils is shown in table 7.4 [102] and Compression index relationships for cohesive soils (Djoenaidi, 1985) is illustrated in figure 7.4 [103].When $(\sigma_v' + \Delta\sigma'_v)$ is larger than pre-consolidation stress, settlement is sum of both virgin compression and recompression portion which is estimated by:

Settlement = [layer thickness × $C_r \times \log_{10}[((\sigma_v' + \Delta\sigma'_{v0})/\sigma'_{v0})/(1 + e_0)]$] +
+ [layer thickness × $C_c \times \log_{10}[((\sigma_{v1}' + \Delta\sigma'_{v1})/\sigma'_{v1})/(1 + e_0)]$

Where C_r is about $0.1\times C_c$ to $0.2\times C_c$.

For multi layers normally consolidated clays the settlement is estimated by:

$$S = \sum \frac{c_c \times H_i}{1 + e_0} \times log \frac{p_{0i} + \Delta p_i}{p_{0i}}$$

According to Mayerhof and Terzagi-Peck formulas, settlement can be calculated as [104]:

$$\Delta H = \frac{q_{allow} \times q_{net}}{N} \qquad \text{(Eq 7.3)}$$

where $q_{net} = q_{allow} - \gamma \times d$, N is average of standard penetration test (N_{30}) numbers in boreholes at corresponding level and d is founding depth.

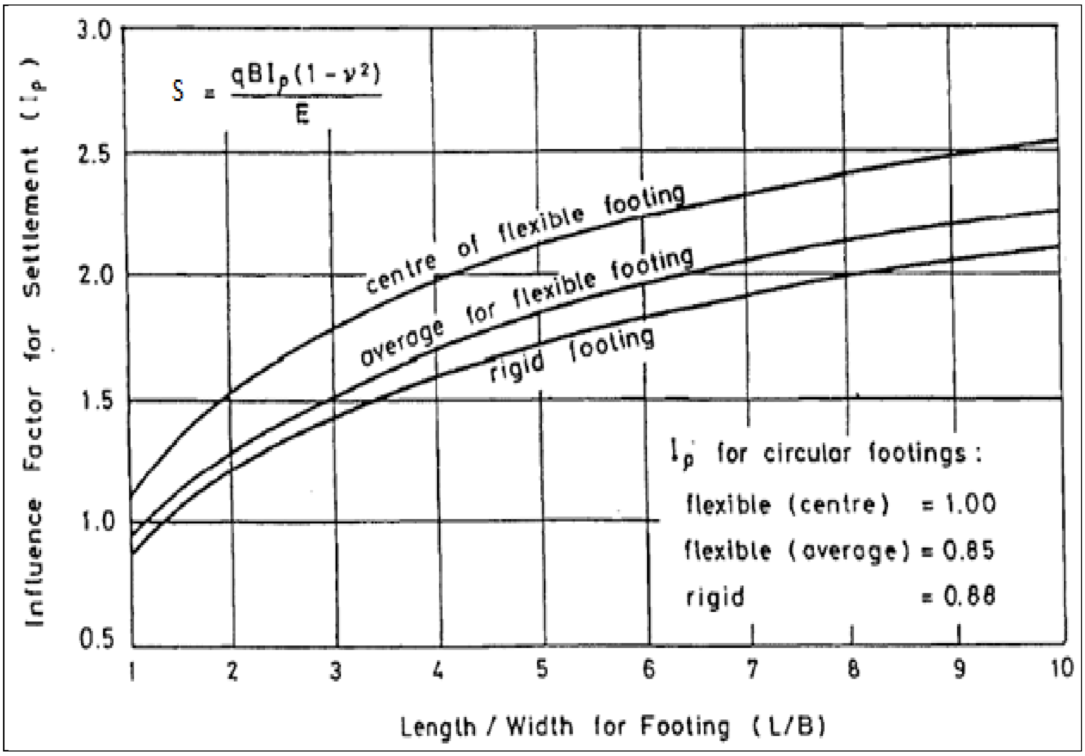

Figure 7.2: Influence factors for settlement of footings on the surface of an elastic solid (after Das, 1984) [88]

Table 7.2: Typical range of Poisons' ratio for different soils [88]

Type of soil	**Poissons′ ratio**
Saturated clay	0.5
Sandy clay	0.3 - 0.4
Unsaturated clay	0.35 - 0.4
Loess	0.44
Silt	0.3 - 0.35
Sand	0.15 - 0.3
Rock	0.1 – 0.4

Table 7.3: Typical range of soil Modulus in undrained state [88]

Ground soil type	**Ground soil Modulus (MPa)**
Very soft clay	0.4 – 3
Soft clay	0.5 – 4
Medium clay	3 – 8.5
Hard clay	1.7 - 7
Sandy clay	28 – 42
Dense sand and gravel	50 – 1000
Weathered rock	200 – 5000
Rock	2000 - 20000

Table 7.4: Typical range of primary compression index (C_c) for different soils [102]

Ground soil type	**Initial compression index**
Rock	0
Dense sand	0.0005 – 0.01
Loose sand	0.025 – 0.05
Hard clay	0.03 – 0.06
Stiff clay	0.05 – 0.15
Medium to soft clay	0.15 – 1.0
Organic soil	1.0 – 4.5

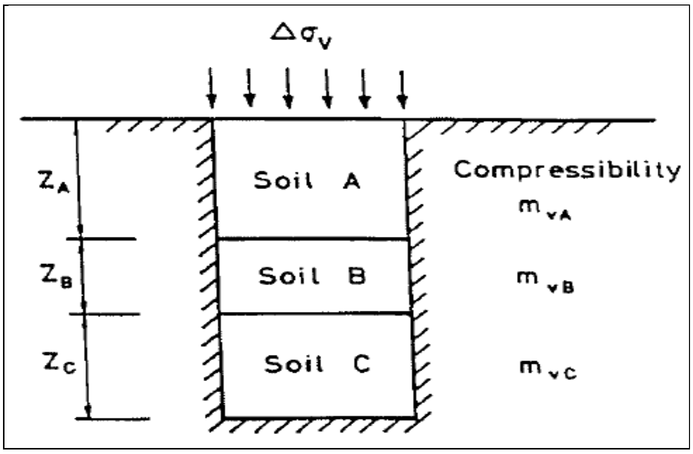

Figure 7.3: Compression of three soil layers [88]

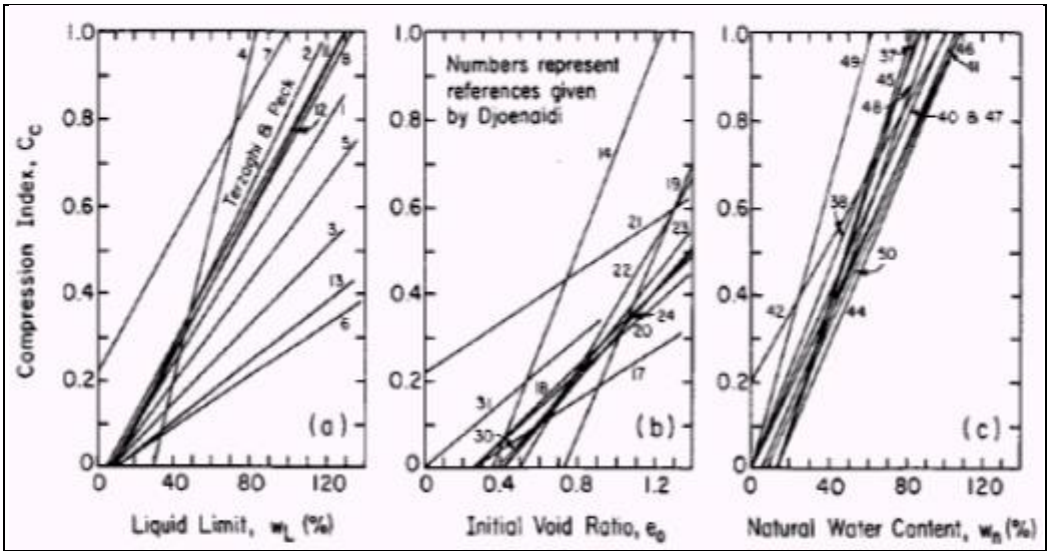

Figure 7.4: Compression index relationships for cohesive soils (Djoenaidi, 1985) [103]

7.3 Adjacent foundations movement due excessive retaining wall deflection

The deformation of retained land is depending on the ratio of maximum deflection of retaining wall versus depth of excavation. Prediction of wall deflection versus surface settlement in each stage of excavation based on early two cases is indicated in figure 7.5 [6] which have measurements for comparing. Peck (1969) prepared an empirical chart to estimating settlements due to deep excavation which is shown in figure 7.6 that gives relationship of Settlement/excavation depth ratio versus distance from excavation / excavation depth ratio in three case of underground soil [10].

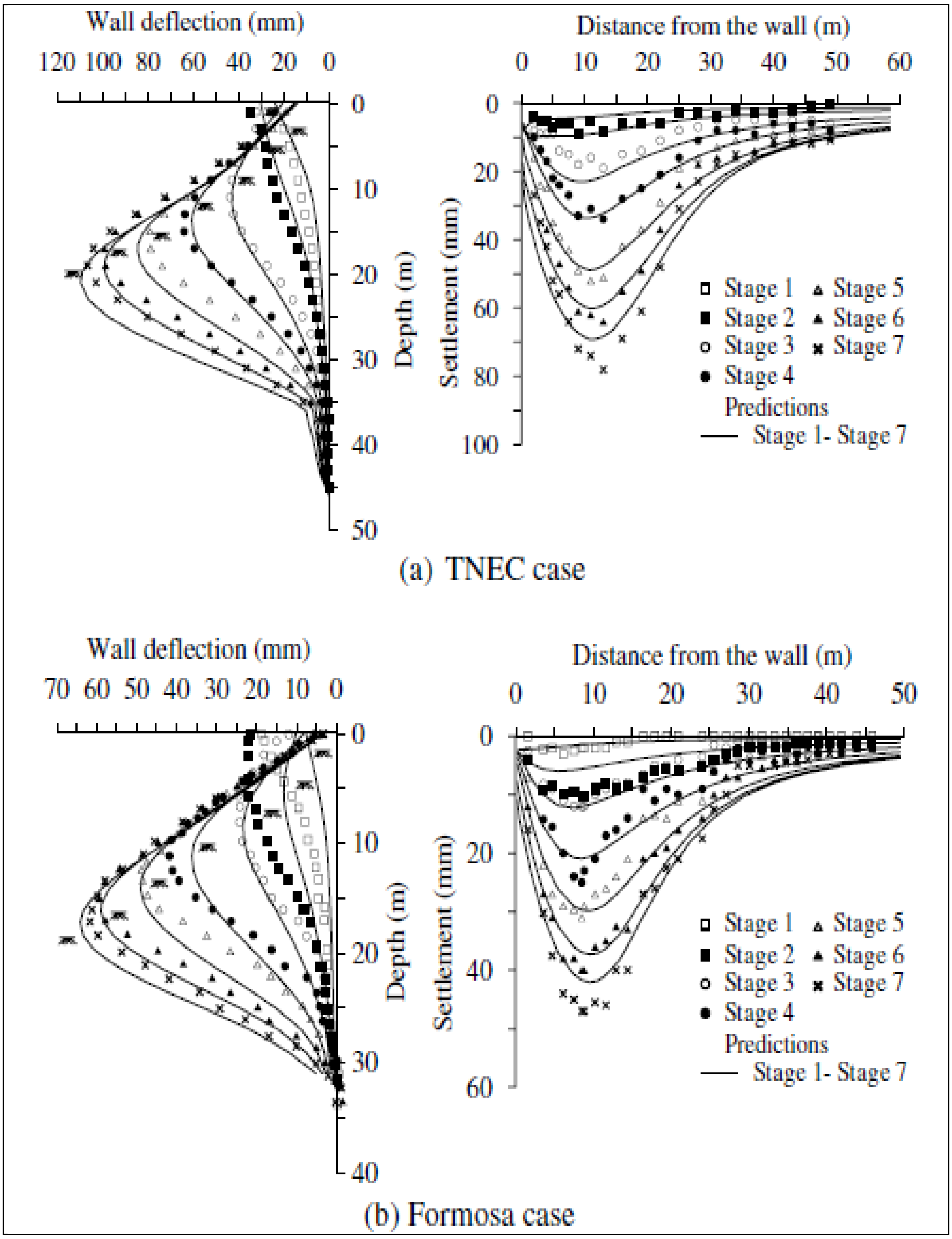

Figure 7.5: Measurements of wall deflection and surface settlement in soft clays and predictions using finite element method with a small-strain soil model [6]

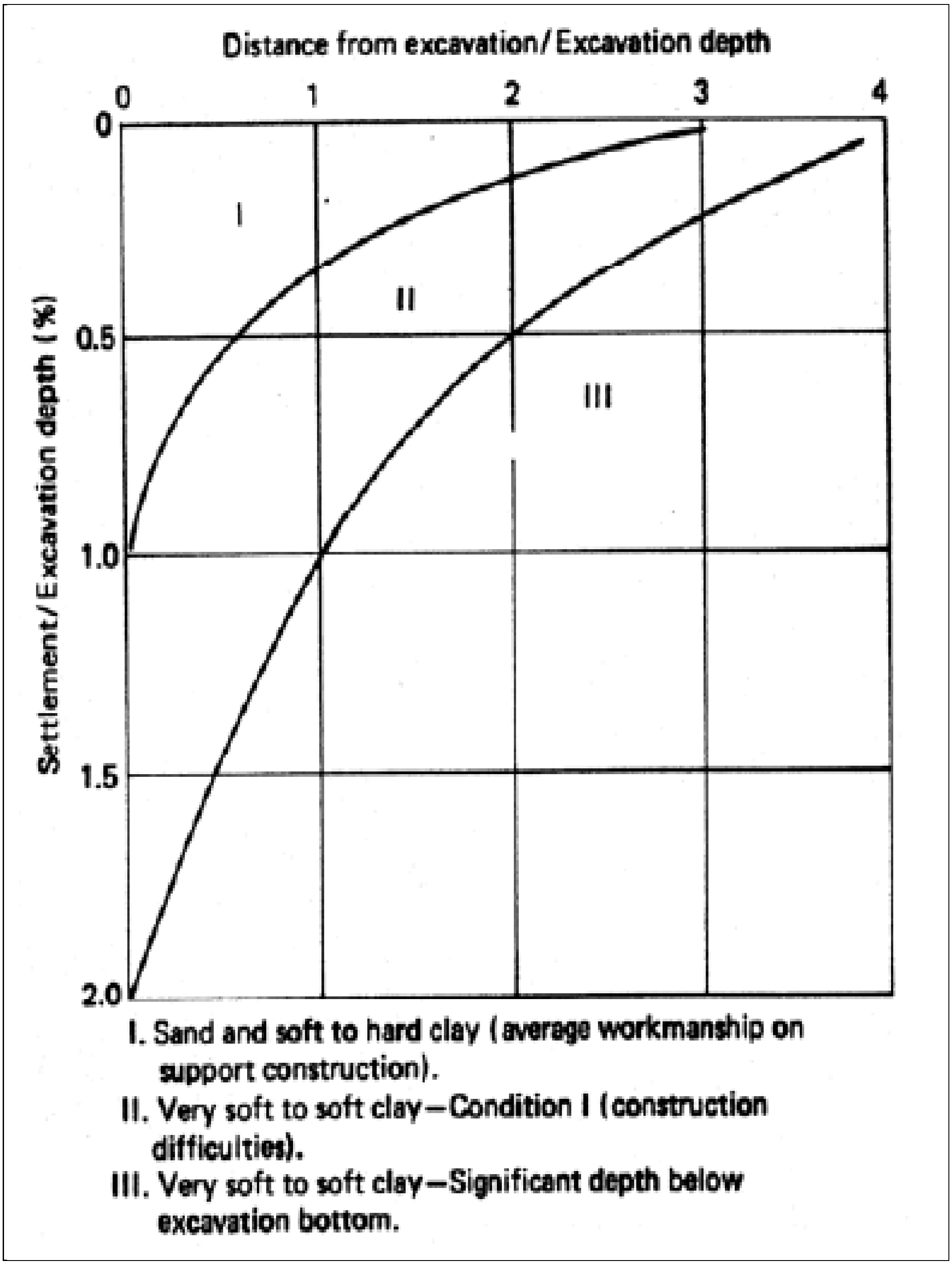

Figure 7.6: (Settlement/excavation depth) versus (distance from excavation/excavation depth) in three case of underground (Peck, 1969) [10]

7.4 Adjacent foundation movement limits

Settlement of foundation or deformation of ground under building is related to subsurface soil type, foundation type, and type of building structure. There are limitations which defined for settlement of structures in building codes or recommendations to prevent damages not only for design but also for respecting the rights of others resulted due to serviceability and safety. Limited movements' classification of foundations of structures as an adjacent building in deep excavation can be based on type of subsurface soil, type of foundation, and type of structure in building codes which are three main source of risk. In other word, building damage depends on underground soil (see chapter 5), foundation type (isolated, mat, pile, etc.), and building structure type such as reinforced concrete frame, steel frame, wood frame, bearing wall, bracing, shear wall, combination of horizontal and vertical resistant structural elements and their substructures characters such as span length, stress-strain modulus, height of columns, beam-column connection, column-foundation connection, cross-section of elements and etc.

The European committee for standardization has recommendation about limiting values of retained land movement for serviceability of adjacent building that is indicated in table 7.5 and for stability in table 7.5 [50]. For example limiting values of total settlement under isolated foundations for serviceability is 25 mm and maximum acceptable movement for total settlement, differential settlement, and angular distortion (rotation) are 50 mm, 20 mm, and 1/500 respectively based on European committee for standardization in European countries [71]. Limiting values of total settlement under raft foundations for serviceability is 50 mm [71]. While the maximum acceptable differential settlement for isolated foundation is 20 mm, the amounts more than zero till fewer than 20 mm produce cracks or large deflection on walls, floors, and differential settlement equal 20 mm is maximum limit that after that there isn't possibility to use it or it is unstable. The European committee recommendation has not point on soil type or range of structure types.

Also System International (SI) guides for allowable uniform settlement, differential settlement, and angular distortion based on kind of structure, kind of shallow foundation, and some kind of soil in comprehensive classification [88]. SI guide for allowable uniform settlement, differential settlement, and angular distortion for structures on isolated foundation on sand and hard clay is shown in table 7.7 [88]. SI guide for allowable uniform settlement, differential settlement, and angular distortion for structures on isolated foundation on plastic clays is shown in table 7.8 [88]. SI guide for allowable uniform and differential settlement and angular distortion for structures on raft foundation on sand and hard clay is shown in table 7.9 [88]. SI guide for allowable uniform and differential settlement and angular distortion for structures on raft foundation on plastic clays is shown in table 7.10 [88]. As it is seen, SI has divided soils into two main groups for settlement and angular distortion issue which are sand and hard clay, and plastic clays. The existence of sand and hard clay in one group may lead to mistakes because sand and hard clay behavior may be differ in different situation for example fluctuated water content, rupture modules, internal friction (ϕ), and cohesion (c_u). Also there isn't any data for other group of soils such as silts or intermediated soils. USSR building code described limited differential settlement for multi story steel or concrete frame structures in clay 1/600 and in sand 1/1000 [105] which has not points for different other cases of structures, other group of soils such as silts or intermediate soils, and more details and identification about mentioned soils. There are other study cases in special locations or special cases without account all effects of soil, structure, and foundation type.

About structural damage occurring due to ground movement, Bjerrum (1963) proposed relationship between potential damage of structure and angular distortion ($\Delta S_T/L$) where ΔS_T is difference in total settlement of two points, and L is distance between two points such as span between two columns. For Brick wall with length/height ratio more than four amount of 1/150 and structures amount of 1/150 have proposed by Bjerrum [88] which remedy may require strengthening the ground, structural jacking, underpinning, and strengthening the structure. Angular distortion of First crack occurrence of interior partitioned wall panel is proposed amount of 1/300 [88].

Except shallow foundation buildings, there can be other cases such as road pavement, retaining wall, deep foundation, drainage, and embankments nearby the deep excavation and may be settled or moved which limiting values of movement is shown in table 7.11 [88].

Table 7.5: Limiting values of retained land movement for serviceability of adjacent building recommended by European committee for standardization [50]

Case	**Differential settlement** (ΔS)	**Total settlement** (S_T)	**Angular distortion** (β)
Isolated foundation		25 mm	1/500
Raft foundation		50 mm	1/500
Frames with rigid cladding	5 mm		1/500
Frames with flexible cladding	10 mm		1/500
Open frames	20 mm		1/500

Table 7.6: Maximum acceptable of retained land movement for stability of adjacent building recommended by European committee for standardization [50]

Case	**Differential settlement** (ΔS)	**Total settlement** (S_T)	**Angular distortion** (β)
Isolated foundation	20 mm	50 mm	1/500

Table 7.7: SI guide for allowable uniform and differential settlement and angular distortion for structures on isolated foundation on sand and hard clay [88]

Case	**Maximum settlement**	**Differential settlement**	**Angular distortion**
Steel structure	50 mm	0.0033×L	1/300
Reinforced concrete structure	50 mm	0.0015×L	1/666
Multi-story RC or steel building with panel walls	60 mm	0.002×L	1/500
Multi-story building, load bearing walls L/H =2	60 mm	0.0002×L	1/5000
Multi-story building, load bearing walls L/H =7	60 mm	0.0004×L	1/2500
Water tower and silos	50 mm	0.0015×L	1/666
L:the length of deflected part of wall, raft or center to center distance between columns H: The height of wall from foundation footing For L/H between 2 and 7 the values are interpolated			

Table 7.8: SI guide for allowable uniform and differential settlement and angular distortion for structures on isolated foundation on plastic clays [88]

Case	**Maximum settlement**	**Differential settlement**	**Angular distortion**
Steel structure	50 mm	0.0033×L	1/300
Reinforced concrete structure	75 mm	0.0015×L	1/666
Multi-story RC or steel building with panel walls	75 mm	0.002×L	1/500
Multi-story building, load bearing walls L/H =2	60 mm	0.0002×L	1/5000
Multi-story building, load bearing walls L/H =7	60 mm	0.0004×L	1/2500
Water tower and silos	75 mm	0.0015×L	1/666
L:the length of deflected part of wall, raft or center to center distance between columns (span) H: The height of wall from foundation footing For L/H between 2 and 7 the values are interpolated			

Table 7.9: SI guide for allowable uniform and differential settlement and angular distortion for structures on raft foundation on sand and hard clay [88]

Case	Maximum settlement	Differential settlement	Angular distortion
Steel structure	75 mm	0.0033×L	1/300
Reinforced concrete structure	75 mm	0.0021×L	1/500
Multi-story RC or steel building with panel walls	75 mm	0.0025×L	1/400
Multi-story building, load bearing walls L/H =2	60 mm	0.0002×L	1/5000
Multi-story building, load bearing walls L/H =7	60 mm	0.0004×L	1/2500
Water tower and silos	100 mm	0.0025×L	1/400
L: the length of deflected part of wall, raft or center to center distance between columns , H: The height of wall from foundation footing For L/H between 2 and 7 the values are interpolated			

Table 7.10: SI guide for allowable uniform and differential settlement and angular distortion for structures on raft foundation on plastic clays [88]

Case	Maximum settlement	Differential settlement	Angular distortion
Steel structure	100 mm	0.0033×L	1/300
Reinforced concrete structure	100 mm	0.002×L	1/500
Multi-story RC or steel building with panel walls	125 mm	0.0033×L	1/300
Multi-story building, load bearing walls L/H =2	60 mm	0.0002×L	1/5000
Multi-story building, load bearing walls L/H =7	60 mm	0.0004×L	1/2500
Water tower and silos	125 mm	0.0025×L	1/400
L:the length of deflected part of wall, raft or center to center distance between columns, H: The height of wall from foundation footing For L/H between 2 and 7 the values are interpolated			

Table 7.11:Limiting values of movement for relevant parameters of other cases[88]

Case	Limit of settlement
Deep foundation	10 mm for Skin friction
Retaining wall	0.1%H for K_a and 1%H for K_p
Reinforced earth wall	25 to 50 for geogrid and 50 to 100 for geotextile
Pavement	20 mm rut depth in major road and 100 mm rut depth in minor road
Embankment	0.1% height of embankment
Drainage	100 to 500 mm

7.5 Building damage classification in term of repairing state

Distinguish between origin of adjacent building damage is important which may be Greenfield and/or inherent settlement effects on structure or very small cracks on interior partition wall due to structure or building construction origin, and/or retained ground deformation or settlement. Greenfield settlement of adjacent building has not relation to retained soil deformation. Also there may be very small cracks on interior partition wall due to structure or construction origin which has not relation to retained soil deformation. Burland et al (1977) condensed numerous approaches to measure building damage based on ease to repair that include three criteria which are: visual appearance, function, and Stability jointly which is indicated in table 7.12 [105]. Building deformation definition (after Burland, 1995) and Schematic diagram of three-stage approach for damage risk evaluation is shown in figure 7.7 [105].

Based on the classification, repair cost of cracks less than 10 mm and angular distortion ($\Delta S_T/L$) less than 1/500 is neglected where ΔS_T is difference in total settlement of two points, and L is distance between two points. Repair cost of cracks less than 5 mm and existence of category 2 is neglected or has slight damage. Repair cost of cracks between 5 and 15 mm and existence of category 3 is moderate. Repair cost of cracks between 15 and 25 mm and existence of category 4 is severe. Repair cost of cracks more than 25 mm and existence of category 5 is very severe.

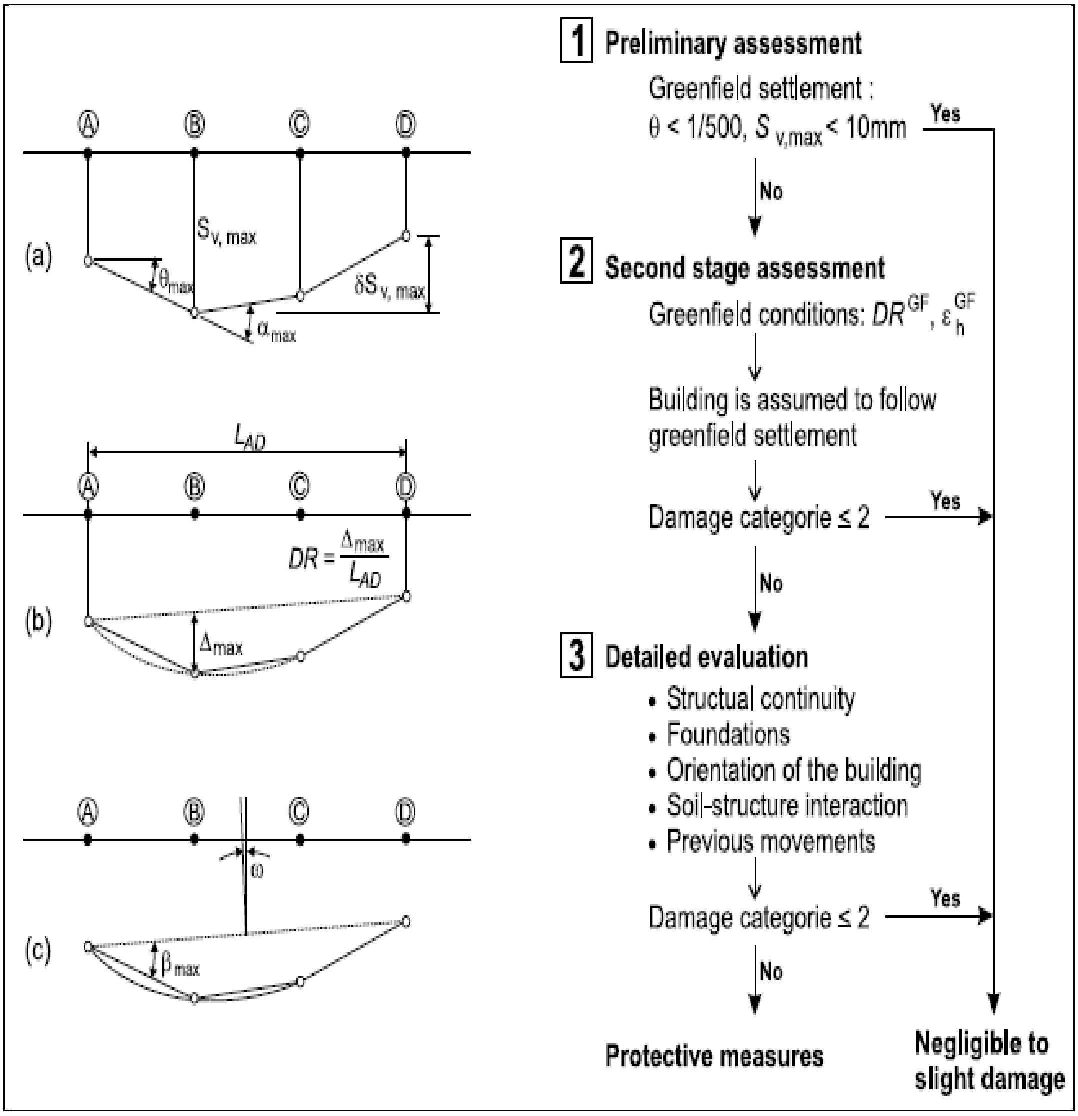

Figure 7.7: Building deformation (after Burland, 1995) definition and Schematic diagram of three-stage approach for damage risk evaluation [105]

Table 7.12: Building damage classification based on repair state [105]

class of damage	Severity level	Damage characterizing	Repair state
0	Negligible	Cracks width ≤ 0.1 mm	No repair
1	Very Slight	Damage normally limited to interior wall finishes. Close examination may disclose a few cracks in outside masonry. Crack widths ≤ 1 mm.	Cracks easily cram through normal decoration.
2	Slight	Cracks width ≤ 5mm Doors and windows might attach a little.	Cracks easily packed. Renovation almost certainly requisite. Recurring cracks can be wearing by appropriate linings. Some Fill in joints of brickwork may be requisite to make sure weather tightness.
3	Moderate	Doors and windows jam. Pipes may break or bent. Weather tightness confused frequently. 5mm < Crack width ≤ 15 mm A few cracks up to 30 mm.	A few opening up in cracks need. Fill in joints of outside brickwork and maybe a small quantity of brickwork to be replaced.
4	Severe	Windows and door frames indistinct, floor inclined clearly. Walls stoop or humping visibly, some loss of attitude in beams. Pipes break. 15mm<Crack width ≤ 25 mm	Wide-ranging repair effort concerning demolishing and replacing segments and components of walls, particularly over doors and windows.
5	Very severe	Beams lose attitude, walls stoop poorly and need shoring. Windows broken with warp. Hazard of instability. Cracks width > 25 mm	Key repair work concerning fractional or whole reconstruction.

7.6 Crack in adjacent lands due to pile driving

Ground shock due to pile driving could be lead to Cracking in nearby lands. Influence effective zone [20] is approximately half of the depth of pile driving (0.5×H). Figure 7.8 shows schematic diagram of this effect.

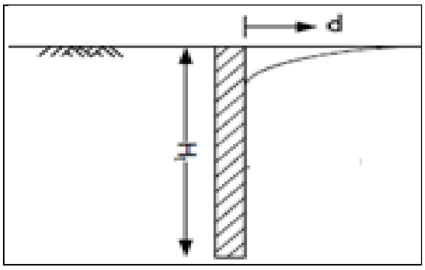

Figure 7.8: Schematic Zone of cracking due to pile driving [20]

7.7 Adjacent land tension cracks during primary excavation

Adjacent tension cracks especially in cohesive soils during primary excavation that depth is lower than 3 meters cause movement in adjacent land, foundation or public utilities. The effective zone is about 0.5 to 0.75 times of excavated depth [20]. Figure 7.9 shows this effect.

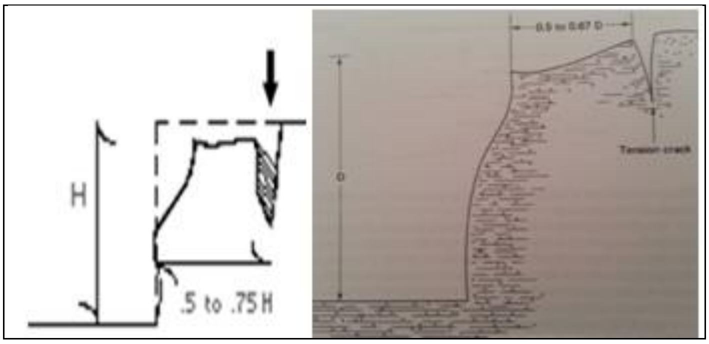

Figure 7.9: Schematic nearby tension cracks during prime excavation and movements [20]

7.8 Subsidence during primary excavation

Subsidence during primary excavation as shown schematically in figure 7.10 due to soil shrinkage in cohesive soils, or due to old in-filled sites and vibration of excavation, and or due to water flow in Non cohesive soils could be occurred [20].

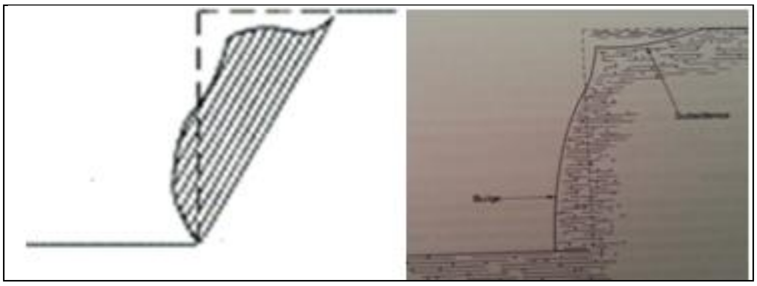

Figure 7.10: Subsidence during primary excavation [20]

7.9 Conclusion

Immediate and consolidated Settlement and related data is gathered and overviewed. The relationship between adjacent foundations movement due extreme retaining wall deflection in soft clay is seen which is from case histories of about 20 meters excavation depth. Key recommendations on nearby building foundation movement limits include settlement (differential, total), and angular distortion is overviewed which must be applied in engineering design depend on location and cited in litigation. Building damage classification due to extreme settlements based on cracks, angular distortion, and repair state is overviewed which is defined as qualitatively.

Chapter 8

DEWATERING

8.1 Introduction

There are some questions such as when and why dewatering is requested in deep excavation and what is the risk of dewatering on deep excavation and nearby lands. The underground which intended to deep excavation is exposure by around surface water. Also it may be underground water which top level is upper than level of depth of excavation. Dewatering in the two mentioned situation is inevitable to mitigate and deal with water existence issue in excavation but it has effects which is reviewed in this chapter.

8.2 Dewatering effects

The traditional method for dewatering is pumping. The Pumping causes decreasing of ground water level which increases the effective stress on soft clay and leads to consolidation settlement. This settlement causes not only settlement of adjacent buildings [106] or facilities but also causes the settlement of base of excavation and leads to decreasing the penetration depth of the retaining wall and start of instability.

The land subsidence caused by dewatering includes three parts: (1) immediate land subsidence which is the subsidence occurring elastic yield during pumping; (2) consolidation of land subsidence which is the condense of soil for steady state after stop pumping and water level keep stable; (3) secondary consolidation land subsidence. Dewatering cause's additional load to soil layers that could be considered as follows [107]:

$\Delta P = \gamma_w \times \Delta h$ (Eq 8.1)

where ΔP is additional load due to dewatering (kPa); Δh is the change of water head in the soil layer before and after dewatering (m); γ_w is water unit weight (kN/m3).

The additional land subsidence caused by dewatering can be evaluated as follows [107]:

$S = \Sigma\ (\Delta P_i \times z_i / E_i)$ (Eq 8.2)

where S is the total additional adjacent land subsidence caused by dewatering (m); ΔPi is the additional load of each soil layer caused by dewatering (kPa); E_i is Young's modulus of the each soil layer in compression (kPa); z_i is the each soil layer thickness (m). For clays and cohesive soils, E_i may be calculated from the compressibility of the soil in each layer as follow [108]:

$E_i = 3(1-2\upsilon\ ) / [[S_{sci} / (\gamma_w \times z_i)] - \lambda \times e_i / (1+e_i)]]$ (Eq 8.3)

where υ is Poisson ratio, S_{sci} is the specific storage coefficient of the ith layer which obtained from the pump testing $(\alpha \times \gamma_w)$, γ_w is water unit weight, z_i is layer thickness, λ is water compressibility (neglected), and e_i is layer void ratio. The specific storage coefficient is equal [109] to $(0.434 \times C_c \times \gamma_w) / [\sigma' \times (1+e_0)]$. Also range of the specific storage coefficient for different type of soils is indicated in table 8.2 [110].

Bottom stability of retained soils exposure to dewatering have to be checked by factor of safety (FOS) which may be calculated as follow:

$FOS = \sigma_v / u$

where σ_v is the total vertical stress and u is the water pressure.

By the corresponding partial factors based on EC7 which recommended 0.9 and 1.35 for the total stress and the pore water pressure, the recommended FOS is obtained as follow:

$0.9 \times \sigma_v = 1.35 \times u$ then: $FOS = \sigma_v / u \geq 1.33$

Figure 8.1 shows fluctuated levels of ground water caused by pumping in different cases such as narrow, and/or width site of excavation, with impermeable layer, with point of pumping as schematically. Degree of pump influencing on ground relates to water discharge quantity [3].

The ground water table is the most unfavorable condition for deep excavation. In several situations with soft clay, the water may be one to two meters below the ground surface. The maximum retaining wall deflection decreases almost linearly with decreasing ground water level [34]. In other word retaining wall deflection has direct relation with water table so that upper water table leads to more deflection of retaining wall.

The Pumping for dewatering influences on soils with permeability between 10^{-6} to 10 which includes very fine sands, silts, clayey silt laminate, desiccated (dried) and fissured clays, clean sands, sand-gravel mixture, and clean gravel. The unfissured clays and well mixed clayey silts containing more than 20% clay are not impacted by pumping commonly [3].

If clayey and silty soils with loss strength get wet by dewatering and discharge, cause expansive case such as swelling and shrinkage [3]. Thus, location of water discharging due dewatering may cause damages for properties or buildings in that ground and it is require to find suitable site without any problem for dewatering water discharge.

In granular soils, the position of the water table is important. Effective stresses in saturated sands can be as much as 50% lower than in dry sand; this affects both the end-bearing and skin-friction capacity of the adjacent building pile which is in granular soils [3].

The occurrence and movement of groundwater affects the carrying capacity and durability of adjacent building piles [106]. An example of dewatering is collected and indicated in table 7.1 and the results are shown in figures 8.2, and 8.3 [106]. As it is seen pump flow rate of 2.4 m^3/h caused 2 mm settlement during 10 days, 10 mm settlement during 100 days, additional 0.01 kPa shear stress during one day, additional 5 kPa shear stress during ten days, additional 1.5 kPa shear stress during 100 days dewatering in sandy clay. Furthermore, dewatering caused 0.01 mm drawdown in one day and one mm drawdown in 50 days for a pile in 3 meters distance from excavation edge. The 10 mm settlement can cause moderate to severe damage on nearby building.

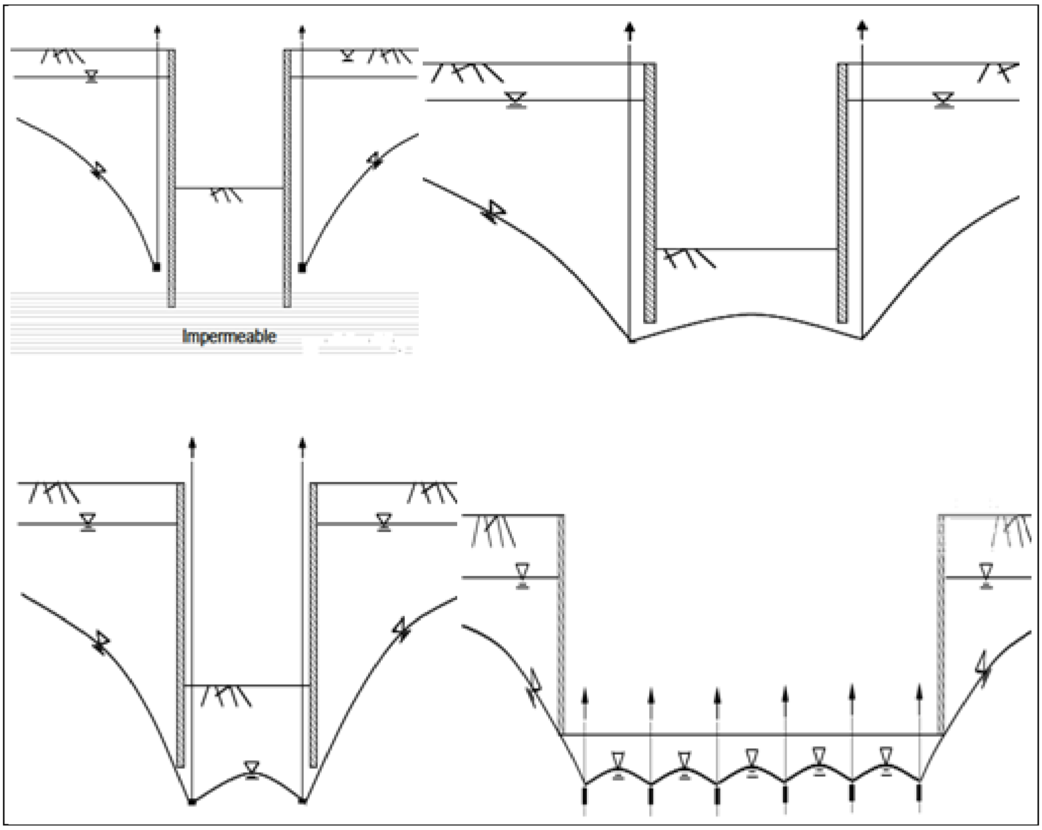

Figure 8.1: Ground water fluctuations due to pumping in soil for dewatering [59]

Table 8.1: A dewatering case history and a pile related factors due to dewatering [106]

Pumping well	**Pile distance from pump**	**Pile dimensions**	**Subsurface**
Perforated length: 3.2 m Flow rate: 2.4 m^3/h	3 m	Diameter:1.5 m Length :12.7 m Reinforce concrete	Phreatic zone with a thickness of 15 m sandy clay

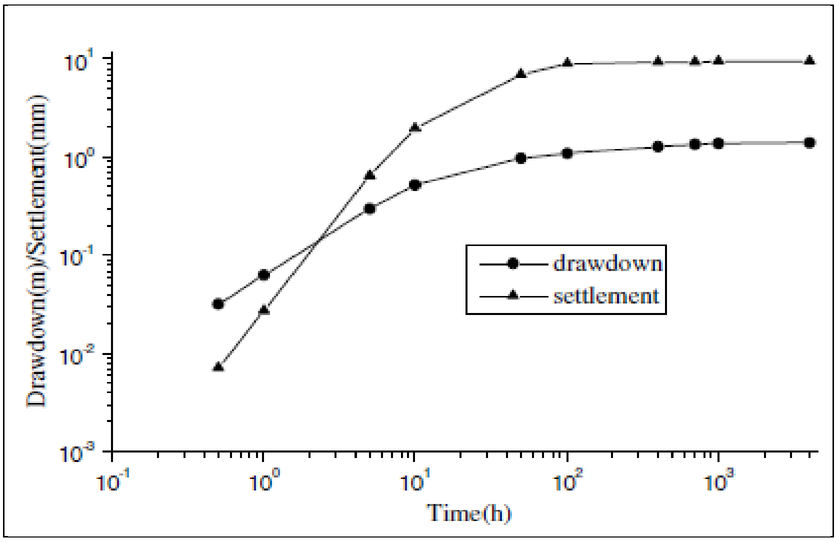

Figure 8.2: Effect of dewatering time on pile settlement in sandy clay (3 m distance) [106]

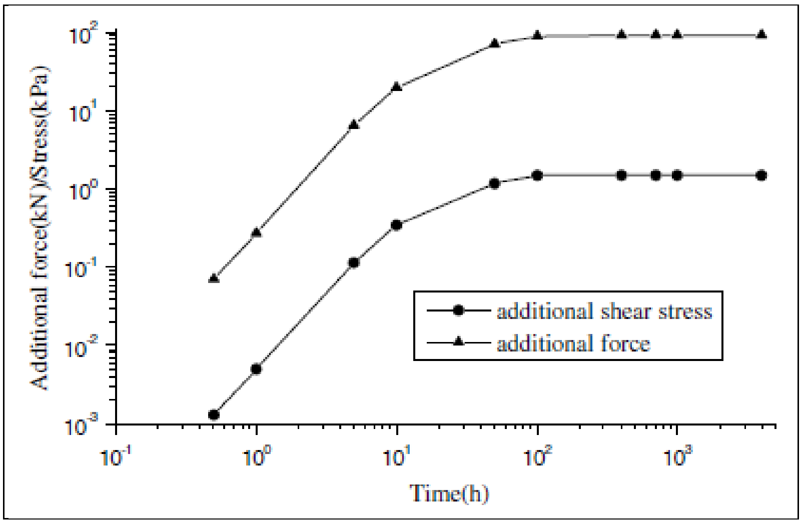

Figure 8.3: Effect of dewatering time on pile additional force in sandy clay (3 m distance) [106]

Table 8.2: The range of specific storage coefficient for soils [110]

Soil	**Specific storage coefficient (m^{-1})**
Plastic clay	2.6×10^{-3} to 2.0×10^{-2}
Stiff clay	1.3×10^{-3} to 2.6×10^{-3}
hard clay	9.2×10^{-4} to 1.3×10^{-3}
Loose sand	4.9×10^{-4} to 1.0×10^{-3}
Dense sand	1.3×10^{-4} to 2.0×10^{-4}
Dense sandy gravel	4.9×10^{-5} to 1.0×10^{-4}
Rock	3.0×10^{-6} to 6.9×10^{-5}

8.3 Conclusion

Dewatering effects on nearby properties such as settlement of adjacent buildings as a risk source which can identify risk is overviewed by recent researches. Dewatering in deep excavation is unfavorable both for adjacent properties and discharge location but it isn't avoiding when water table level is higher than excavation depth.

A case history of dewatering is collected in which the pump flow rate of 2.4 m^3/h caused 2 mm settlement during 10 days, 10 mm settlement during 100 days, additional 0.01 kPa shear stress during one day, additional 5 kPa shear stress during ten days, additional 1.5 kPa shear stress during 100 days dewatering in sandy clay.

Chapter 9

ILLUSTRATIVE EXAMPLES, CONSIDERATIONS, AND CASE STUDY

9.1Introduction

Dealing with risk of underground soil types of soft clay and dense sand for 16 m excavation depth supported by cantilever diaphragm wall is designed and solved in illustrate examples 9.2, 9.3, and 9.3 which cantilever diaphragm wall length has been differed significantly.

Potential damages due to sliding in the cases of before, during and after diaphragm wall constructing is surveyed in illustrate examples 9.4 which can identified risk of construction processes risks.

A top-down method after construction of reinforced concrete diaphragm wall and pile foundation for frame and slabs from case histories is used to geotechnical risk assessment based on proposed method and also a risk response plan is proposed for that case in illustrate examples 9.6. Based on geometry of site and geotechnical investigation which are soil appearance and standard penetration tests results in each layer, sensitivity analysis is done and maximum probability of risk occurrence in each stage of excavation is estimated which has a proposed formulas. Then risk consequence estimation is done.

A case study from North Cyprus is presented in illustrative example 9.7 for clarifying and estimating cost risk due to delay, material price increase, material additional consumes, and adjacent buildings and lands settlements.

Considerations on situation of some case histories and some others case studies in deep excavation are presented in 9.8 which briefly explains the situation of cases and commands.

Considerations on risk assessment of production rate and duration in case of grab operation, excavating operation, and hauling operation processes is overviewed in 9.9 and range of unit cost for some case histories with different geometry and size of project is estimated. Also recommendations are proposed for deep excavation contract risks.

In this chapter there are risk identifications such as:

1- Geotechnical risk for example sliding, overturning, Basel heave, bottom heave due to unloading, heaven due to artesian pressure, and piping.
2- Cost risk for example delay, material additional consuming, material price increase, nearby buildings and road settlement risk, contract conditions risk, and budgeting risk
3- Working condition risk for instance weather condition risk, maneuvering risk, mechanical breakdowns risk, operator efficiency risk, and waiting for dump trucks risk

9.2 Cantilever retaining wall length require to mitigate sliding in soft clay

Let us assume the ground is soft clay, γ_{soil} =15.6 kN/m^3, K_a=0.5, c_u=12 kN/m^2, excavation depth =16m (since the 4 story underground), and reinforced concrete wall tanδ = 0.50777, out of earthquake zone, find the minimum required wall length due to sliding.

Solution:

$\sum F_S = \sum P_{active} = 50\times(16+h)+3.9(16+h)^2-16.97056(16+h) = 1526.87+157.82944h+3.9h^2$

Resistance to sliding = $\sum F_R = R \tan\delta + C_b B + \sum P_{passive}$

$\sum F_R = (1526.87+157.82944h+3.9h^2)\ 0.50777+12B+9.75h^2+26.832815h$

$\sum F_R = 775.299+106.973815h+11.7303h^2+12B$

If it is permanent wall [86]: $\sum F_R = 1.75 \sum F_S$

$775.299+106.973815h+11.7303h^2+12B = 1.75(1526.87+157.82944h+3.9h^2)$

$4.9053\ h^2 - 169.227705h - 1896.7235 = 0$ (12B is neglected)

$h = [169.227705+ (169.227705^2+ 4\times4.9053\times1896.7235)^{0.5}] / (2\times4.9053) = 43.4$ m

Wall length = 43.4+16 = 59.4 m

If it is temporary wall [86]: $\sum F_R=1.25\sum F_S$

$775.299+106.973815h+11.7303h^2+12B = 1.25(1526.87+157.82944h+3.9h^2)$

$6.8553\ h^2 - 90.313h -1133.2885 = 0$ (12B is neglected)

$h = [90.313+ (90.313^2+ 4\times6.8553\times1133.2885)^{0.5}] / (2\times6.8553) = 21.03$ m

Wall length = 21.03+16 = 37.1 m

9.3 Cantilever retaining wall length require to mitigate sliding in dense sand

Let us assume the ground is dense sand, γ_{soil} =18.3 kN/m^3, excavation depth =16m (since the 4 story underground), K_a = 0.19823, and reinforced concrete wall tanδ = 0.57735, out of earthquake zone, find the minimum required wall length due to sliding.

Solution:

$\sum F_S = \sum P_{active} = 19.823H + 1.8138045H^2 = 19.823\times(16+h) + 1.8138045(16+h)^2$

$\sum F_S = 317.168+19.823\times h+464.334+58.0417h+1.8138045h^2$
$= 1.8138045h^2+77.8647h+781.5$

$\sum F_R$ = Resistance to sliding = $R \tan \delta + \sum P_{passive}$

$\sum F_R = (1.8138045h^2+77.8647h+781.5) \times 0.57735 + 46.158456\ h^2$

If it is permanent wall [86]: $\sum F_R = 1.75 \sum F_S$

$47.2056h^2+44.95478h+451.199 = 1.75(1.8138045h^2+77.8647h+781.5)$

$44.03h^2-91.308h-916.426 = 0$

h=5.72 m

Wall length = 5.72+16 = 21.72 m

If it is temporary wall [86]: $\sum F_R = 1.25 \sum F_S$

$47.2056h^2+44.95478h+451.199 = 1.25(1.8138045h^2+77.8647h+781.5)$

$44.9386h^2 - 52.37622h – 525.676 = 0$

h= 4.05 m

Wall length = 4.05+16 = 20.10 m

9.4 Sliding risk potential due to project underground soil type

The comparison of the two above examples indicates that the Sliding plays a decisive role on wall length and penetration selection. Also the kind of soil can influence the wall length intensively so that in equal geometry condition (e.g. 16 m excavation depth), the length of permanent reinforced concrete wall should be 59.4 m, and temporary wall 37.1 m for soft clay, while for dense sand, the length of permanent reinforced concrete wall should be 21.72 m, and temporary wall 20.10 m. The other point is the influence of the kind of soil on investigation depth so that in soft clay by groundwater level, there is need to reach to more than 59.4 meters depth of investigation ($3.7 \times h_{excavation}$) while in dense sand there is need to reach to more than 21.72 m ($1.4 \times h_{excavation}$).

Note: Use of more passive load, more anchoring, increasing the width of wall, increasing the penetration of wall, and use of strut supporting system are several alternatives to grow stability against sliding.

9.5 Potential damages due to sliding in case of diaphragm wall constructing

●Before diaphragm wall implementation: After fixing of alignment, guide wall construction, and during trenching by hydraulic excavator-backhoe, there isn't important damage often because the trench depth is about 1.5 meters that is the safe height. Trench for diaphragm wall is implemented in length of 4 - 6 meters by clamshell. For examples 9.2 and 9.3, the depth of trench excavation are 20.1 to 59.4 depend on kind of underground soil and temporary or permanent function of support that bentonite slurry is used for preventing of sides collapsing. However, it is possible that the sides of excavated trench are collapsed. In that case maximum underground collapsed volume could be normally multiplication of width of trench to half of height of excavated trench unless there are covens in underground with availability together and in this case bentonite is lost and trench is collapsed [111]. If due to error the trench depth is increased in this case probability of damage occurrence could increase.

●During diaphragm wall implementation: During wall implementation there are cases such as trench is not reach to final depth, trench is completed but cages were not installed, cages were installed but concreting is not started, cages were installed and concreting is started but not finished.

●After diaphragm wall implementation: in that case collapse could occur during excavation or during excavation stages and secondary supports installation so that the excavated depth is more than plan depth and passive force is reduced. Passive force is highly sensitive to penetration of wall under final depth of excavation. Also if the wall length is implemented less than plan it will lead to decreasing passive force and increasing the risk probability and damage. The idealized shape of sliding could be assumed as the figure 9.1. Occurrence of sliding causes damage on wall, excavated lot with probable existence of machines and workers, and adjacent land with all additions such as existent buildings, urban facilities, cars and pedestrians.

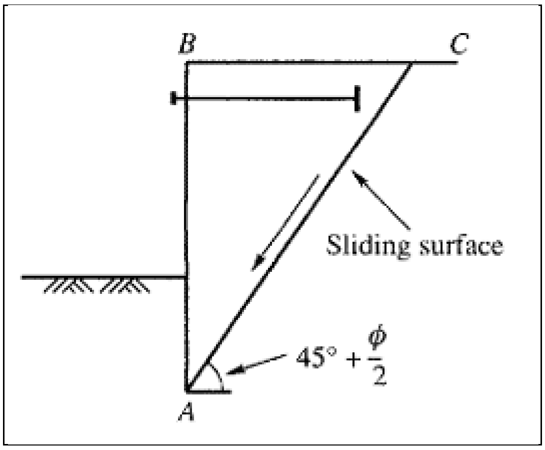

Figure 9.1: An idealized shape of sliding after wall installation during excavation stages [66, 67]

9.6 A Top-down method risk assessment and management

A top-down method after construction of reinforced concrete diaphragm wall (0.9 m thickness, and 35 m length and 192 m perimeter) and pile foundation implementation for frame and slabs with monitoring system in very soft clay (CL, N=2 to 5 SPT number) with high level water table is presented for a national enterprise center with area about 1800 m^2 relatively irregular shape [7]. Underground condition and arrangement of instrumentation is shown in figure 9.2 [7]. Operation stages are indicated in table 9.1. In all operation stages, the wall deflection and adjacent earth settlements is monitored by instruments and the results are shown in figure 9.3 [7]. Let assess geotechnical risk of deep excavation which include geotechnical risk identification such as sliding, overturning and so on, estimating risk occurrence probability, and estimating risk consequence which is expected damages in construction. Let assume five basement underground and ground level plus seven stories each with 1800 m^2 building area and for 192 meters perimeter neighborhood, 24 meters wide of road with water supply pipes, electric, telecommunication cables in pipes and then eight story reinforced concrete residential buildings as adjacent private or public properties.

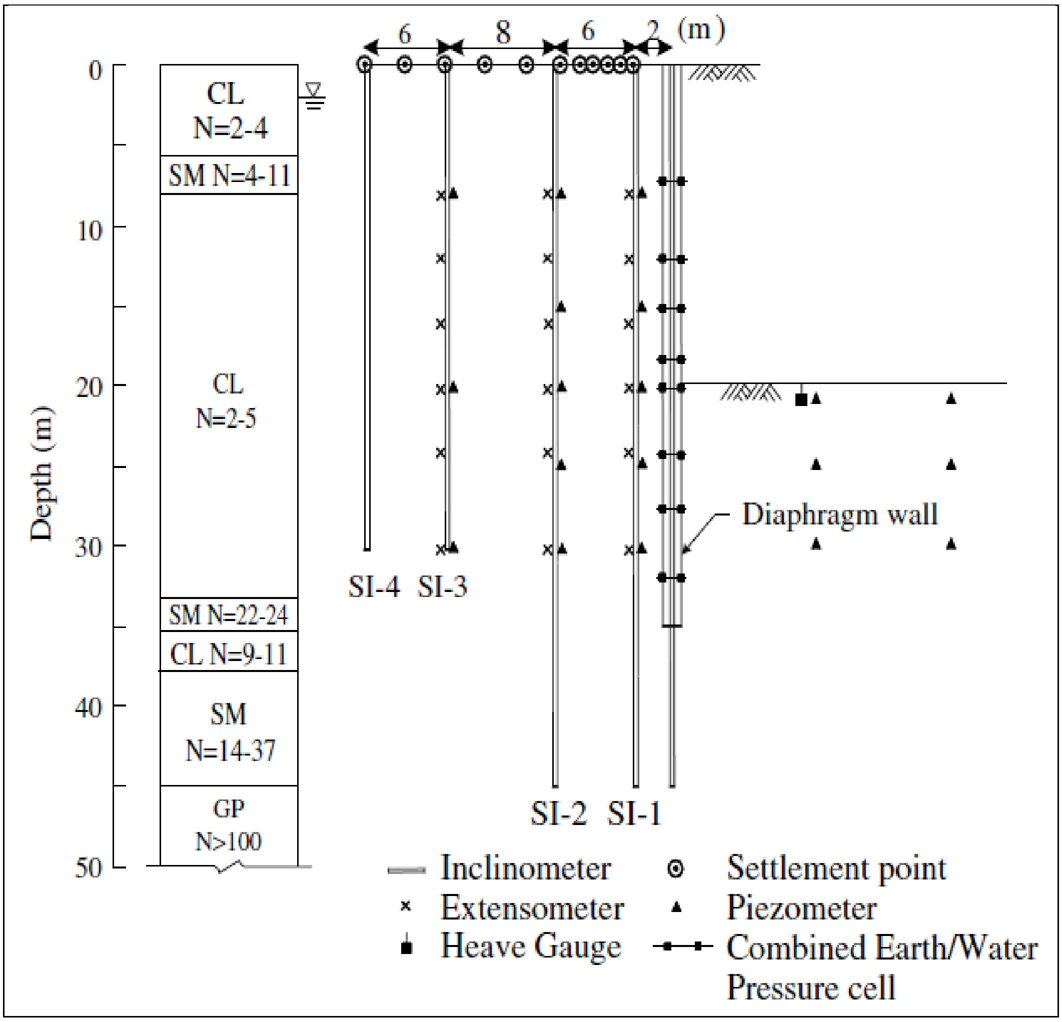

Figure 9.2: Underground condition and arrangement of instrumentation [7]

Table 9.1: The top-down method operation stages of example [7]

Operation stages after construction of diaphragm wall and pile foundation
1 Excavation to elevation -2.8 m
2 Steel flying shore 15H300×300×10 at -2.0 level, preloaded
3 Excavation to elevation -4.9 m
4-1 Mold floor slab (0.15 m) at elevation -3.5 m
4-2 Demolished -2.0 level prop and -3.5 level cast
5 Excavation to elevation -8.6 m
6 Mold floor slab (0.15 m) at elevation -7.1 m
7 Excavation to elevation -11.8 m
8 Mold floor slab (0.15 m) at elevation -10.3 m
9 Excavation to elevation -15.2 m
10 Mold floor slab (0.15 m) at elevation -13.7 m
11 Excavation to elevation -17.3 m
12 Steel flying shoe 21H400×400 ×13 at -16.5 level, preloaded
13 Excavation to elevation -19.7 m
14 Mold foundation slab
15 Mold floor slab (0.15 m) at elevation -17.1 m
16 Demolished -16.5 level of prop

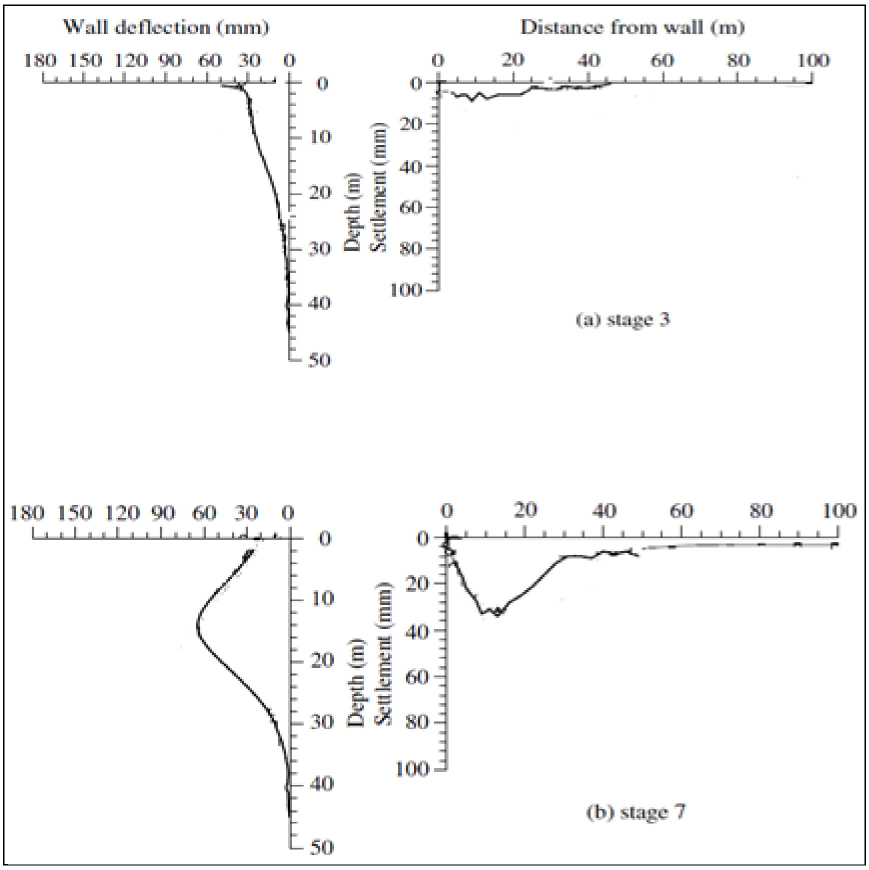

Figure 9.3: Wall deflection and adjacent earth settlements in different stages [7]

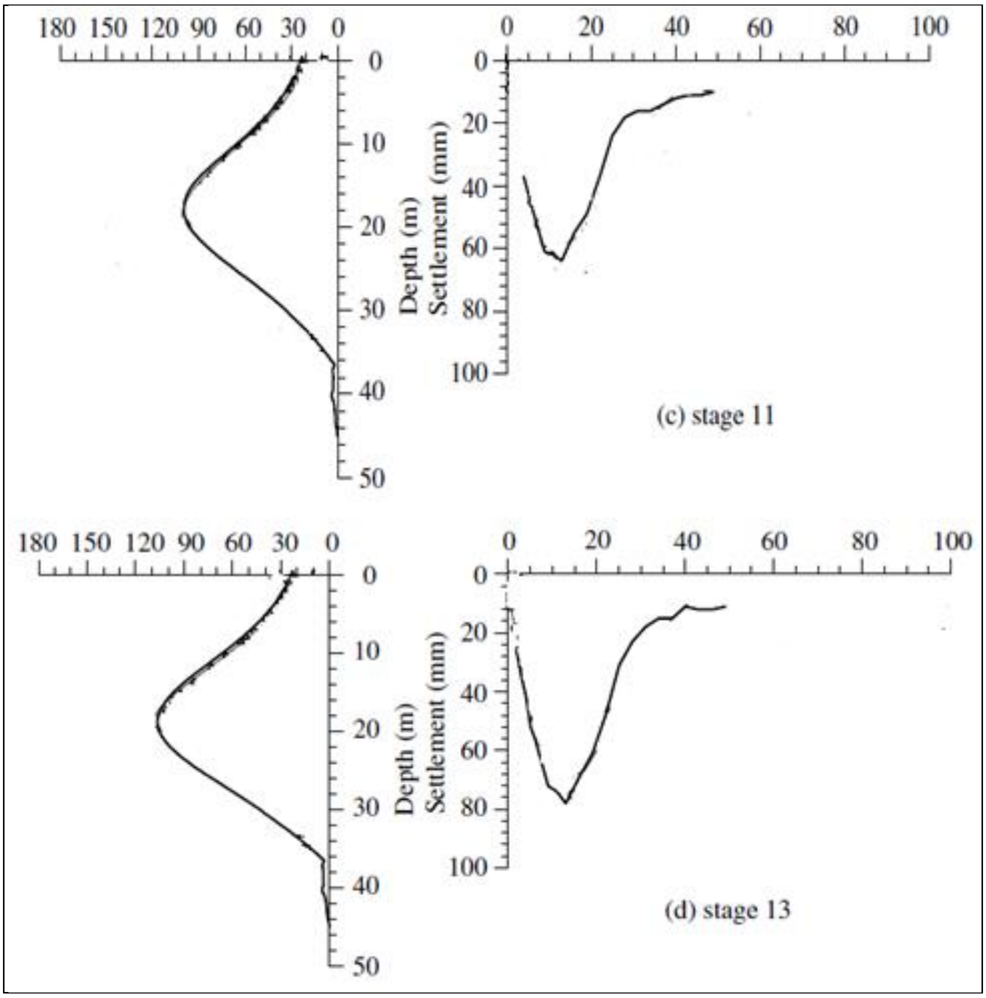

Figure 9.3 Wall deflection and adjacent earth settlements in different stages (continued) [7]

9.6.1Briefly description of situation

Among operation stages, the seven stages are for excavation. Underground condition shows SM (N=22 to 24) layer with 2 meters thickness from -33.3 level and under that stiffer clay (CL, N=9 to 11) with 2.5 meters thickness and under that SM (N=14 to 37) with 7.5 meters thickness and under that GP (N>100).

The wall deflection and adjacent earth settlements has measured with monitoring. The maximum excavation depth of 19.7 meters was been excavated in seven stages. After finishing the final stage of excavation, at distance of 14 meters, the maximum settlement is about 79 mm and wall maximum deflection is about 110 mm. Also at distance of 35 meters, the maximum settlement is about 16 mm. Even in 4th stage of excavation (7) the settlement is 32 mm at distance 10 – 14 m. If there had adjacent building with isolated foundation, the existence settlement in 4th stage excavation were damaged it (32mm>25mm) very severe so that key repair work concerning fractional or whole reconstruction is required. However risk is depending on existent building value (without land value) and its distance.

9.6.2 Situation study in comparison with failure modes for uncertainty and risk analysis

Sliding analysis for low and high ranges of soil parameters (γ_{soil}, c_u, K_a, K_p) and adhesion (C_b) based on reported underground investigation is done which is indicated in table 9.2. The use of range of soil parameters is due to uncertainty. Summary of results of probability occurrence for sliding is indicated in table 9.3 (recommended Permanent: $FOS_{sure} \geq 1.75$, temporary: $FOS_{sure} \geq 1.25$ [86]).

Possible internal damages due to sliding are: volume of soil in sliding, demolished slider reinforced concrete (RC) diaphragm wall, RC diaphragm wall reconstruction, early works, early overhead, and equipment.

The volume of soil in sliding is calculated based on probable shape of sliding which shows in figures 9.3 and 9.1 that for soft clay the ϕ is about zero. The cost for demolishing slider reinforced concrete (RC) diaphragm wall assumed the daily wage of three workers per one cubic meter of reinforced concrete [112] which its volume obtain by multiplication of 0.9 (wall thickness in meter) to excavation depth in each stage of excavation in unit wide of perimeter of retaining wall. The RC diaphragm wall reconstruction is estimated with assuming $50 for one cubic meter concrete (in situ) and $15 wage of concreting (crane for tremie pipe and head fixing/movement, concrete workers), 230 kg steel reinforcement (with waste) per square meter of wall which price (buy, transport, discharge) is $400 per ton (metric) and reinforcement wage (including inserting steel cage with crane) is $100 per ton in each stages excavation depth, 2.9 square meter formwork for one square meter of wall which wage (with material) is $10 per square meter in each stages excavation depth. The early works is estimated by foreseeing grab trenching (including guide wall), reinforced concrete wall implementing, and upper stages excavation so that hourly rate of 5.0 LCY (3.84 LCM-heaped) grab (bucket fill factor 0.95) is assumed $11.25, crawler clamshell hourly rate is assumed $121 with estimated 45 second cycle time and 0.65 job efficiency and 90% job management which average volume of trenching production is assumed 189 m^3 per hour, 230 kg steel reinforcement (with waste) per square meter of wall which price is $400 per ton (metric) and wage is assumed $100 per ton, $50 for one cubic meter concrete (in situ) and $15 wage of concreting (crane for tremie pipe and head fixing/movement, concrete workers), $3.5 per in-site cubic meter upper excavation, guide wall 0.35 meter thickness break wall with cement mortar in 1.5 meters height in two side of trench (each break:$0.035, and one square meter break wall includes 600 break, wage $121 per cubic meter break wall, mortar: $60 per cubic meter which consumed 0.3 for one cubic meter of wall, are assumed. Overhead is assumed 20% of early works, and equipment is assumed 3% of early works. Result of Possible internal damages due to sliding is estimated in term of $ per meter of site perimeter and also expected internal damages based on probability of failure (sliding in each stage of excavation) occurrence is indicated in table 9.4.

Possible external damages due to sliding are: road, and adjacent buildings. Road full depth patching costs include [113]: human resource \$11.186 per square meter, machinery \$6.48 per square meter, material \$58.55 per square meter (water \$20 per m3, gravel for grading \$45 per m3, binder aggregate \$45 per m3, bitumen 60-70: 1.34 per kg, mc2 bitumen \$1.34 per kg, gasoline \$3.1 per liter), and transportation \$0.59 per square meter which in total is \$77 per square meter of asphalt road. For roadbed another utilities such as electric or telecommunication cable or water supply \$20 per square is assumed. Damage of adjacent building depends on distance and ground settlements. Foundation settlement in primary steps (e.g. 5 to 15 mm) causes cracks in non-load-bearing brickworks which destroy plasters and paints, imbalance of doors and windows, dislocation of installation, if we assume \$550 per square meter for building area construction cost (BACC), in 3rd stage excavation (depth=8.6) 3% BACC for rehabilitation cost of building at 20 meters distance in 2 meters dimension of adjacent building is seen enough, in 4th stage excavation (depth=11.8) 15% BACC (more than 28 mm settlement) for rehabilitation cost of building at 20 meters distance in 5 meters dimension of adjacent building is seen adequate, in 5th stage excavation (depth=15.2) 25% BACC for rehabilitation cost of building at 20 meters distance in 10 meters dimension of adjacent building is seen sufficient, in 6th stage excavation (depth=17.3) 35% BACC (more than 48 mm settlement) for rehabilitation cost of building at 20 meters distance and 20 meters dimension of adjacent building is seen enough if exists, and for 7th stage excavation (depth=19.7) 75% BACC cost (about 60 mm settlement) for rehabilitation of building at 20 meters distance and 25 meters dimension of adjacent building is seen enough if exists. Possible external damages and expected external damage due sliding is indicated in table 9.5. Based on result, the share of internal damages at stages 3rd, 4th, 5th, 6th, and 7th of excavation are 86.64%, 67.23%, 47.93%, 28.49%, and 14.84% respectively. This shows that at excavation stages from top to bottom, the share of internal damages are reduced. Figure 9.4 shows relationship between internal damage share percent and excavation stage from top to bottom for sliding. Also based on result, the share of external damages at stages 3rd, 4th, 5th, 6th, and 7th of excavation are 13.36%, 32.77%, 52.08%, 71.51%, 85.16% in that order. This represents that at excavation stages from top to bottom, the share of external damages are increased. Figure 9.5 shows external damage share percent in term of excavation stage for sliding. Expected damages due sliding and its impact on project (with assuming $550 per square meter building construction cost) are indicated in table 9.6.

Because of probability influence on action, resistant, and FOS determining, a tolerance could be considered for cases of optimistic and pessimistic FOS determining. Tolerance depends on abundance of geotechnical investigation tests but considering EC7 recommendations on site investigation it seems 10% tolerance could appropriate because of EC7 recommendation on optional increasing 10% on retaining wall calculated length due to possible over-excavation as resulted error in depth [34]. Optimistic, normal, and Pessimistic expected damages and their impact on project due to sliding after diaphragm wall construction is shown in table 9.7.

To deal with the sliding risk, secondary support or multi-level support that can carry out the request additional lateral force against actions is need to increase FOS_{min} up to reliable number. In this example pre-loaded steel struts and floor slabs are acted as resistant against actions for increasing FOS_{min} of sliding up to 1.75 for slabs as permanent and up to 1.25 for steel struts as temporary supports that are indicated in table 9.8. The required area due to sliding for steel struts is based on building steel ultimate strength (e.g. 4200 kg/cm^2) and it is necessary it checks for bending due its self weight, slenderness, and connection to retaining wall. Also it is require that area is compared and checked with other factors requirements such as overturning, etc.

Overturning analysis for low and high ranges of soil parameters (γ_{soil}, c_u, K_p, K_a) based on reported investigation is shown in Table 9.9. Summary of results of probability occurrence for overturning failure is indicated in table 9.10 (recommended Permanent: $FOS_{sure} \geq 2$, temporary: $FOS_{sure} \geq 1.25$ [83]).To deal with the risk the resistant forces needs to increase FOS_{min} of overturning to reach sure case. Resistant request by struts and slabs for increasing FOS_{min} of overturning up to 2 for slabs and up to 1.25 for steel struts is shown in table 9.11.

Basel heave analysis for low and high ranges of soil parameters (γ_{soil}, c_u) based on reported investigation is indicated in table 9.12 (recommended FOS $\geq$ 1.2 [83]). There could be Basal heave in stages 1st until 7th of excavation. Probability of failure occurrence is one (certainly) for all stages except fourth stage of excavation. That heaves is removed in next stage of excavation which slightly increases the excavation volume and causes slightly deformation of nearby lands and settlement under adjacent foundation. For reducing or even eliminating the heave, reducing of surcharge (from 100 kN/m^2 to 13 kN/m^2) is effective if it is possible in practice. The result of surcharge reducing is shown in table 9.13. Main foundation's weight must raise FOS up to1.2 for case of surcharge existence.

Bottom heave due unloading analysis for low and high ranges of soil parameters (γ_{soil}, c_u) based on reported investigation is indicated in table 9.14 (recommended

FOS ≥ 2 [86]). There could be Bottom heave due unloading in stages 1st until 7th of excavation. That heaves is removed in next stage of excavation which slightly increases the excavation volume and causes deformation of nearby lands and settlement under adjacent foundation. If depth of excavation is bigger than wide of excavation it is possible to reduce or even eliminate (in some cases) the heave by reducing surcharge (to a certain extent), otherwise it is inevitable or large loading is required. Large loading is expected after construction of building then at the time of excavation execution it is inevitable and should be accepted or discouraged the project.

Heaven due artesian pressure analysis for low and high ranges of soil parameters (γ_{soil}) based on reported investigation is shown in table 9.15. There could be heaven due artesian pressure in stages 4th until 7th of excavation. Summary of results of probability occurrence of artesian heaven failure is indicated in table 9.16 (recommended FOS ≥ 1.2 [86]). That heaven is removed in next stage of excavation which slightly increases the excavation volume. It is inevitable at the time of excavation execution and its back expected after construction of main foundation or even building and should be accepted or discouraged the project.

There is not any bottom impermeable layer, thus there isn't upheaval failure. Also there is not loose sandy soil, thus there isn't sand boiling failure, and liquefaction too.

Hydraulic failure analyzing based on Terzaghi method for low and high ranges of soil parameters based on reported investigation is indicated in table 9.17 (recommended Permanent: FOS$\geq$1.2 , temporary: FOS$\geq$1.5 [86]). Also Hydraulic failure analyzing based on critical hydraulic gradient method for low and high ranges of soil parameters based on reported investigation is shown in table 9.18 (FOS≥2 [86]) There could be hydraulic failure in stages 5th until 7th of excavation. To mitigate and deal with the hydraulic failure risk there is need to dewatering before excavation (even a few months in clays) and during excavation so that the water table comes under excavation level.

Dewatering consequences is settlement of soils under adjacent foundation or utilities because pore water is discharged and pore pressure converts to near zero and due to early loads soil particles position changes to new constitute and settlement or even soil deformation is occurred. Without dewatering, hydraulic failure is inevitable at the time of excavation execution and it is expected after construction of main foundation or even building and should be accepted or discouraged the project unless freeze of pore water which may have heaven effects.

Table 9.2: Sliding analyses without struts and slab for low and high ranges of soil parameters based on reported investigation

Excav depth	γ_{soil}	$\mathbf{c}_u$	$\mathbf{K}_a$	$\mathbf{K}_p$	$\mathbf{q}_{sur}$	$\sum P_{passive}$	$\sum P_{active}$	R tan δ	$\mathbf{C}_b\mathbf{B}$	**FOS**
2.8	15.6	12	0.5	2	100	23838	5862	2976	10.8	4.58
2.8	15.6	24	0.5	2	100	24969	5247	2664	10.8	5.27
2.8	15.6	12	0.8	1.25	100	15275	9451	5271	10.8	2.18
2.8	15.6	24	0.8	1.25	100	16170	8673	4837	10.8	2.42
2.8	17.8	12	0.5	2	100	26119	6536	3318	10.8	4.51
2.8	17.8	24	0.5	2	100	27250	5921	3006	10.8	5.11
2.8	17.8	12	0.8	1.25	100	16701	10529	5872	10.8	2.14
2.8	17.8	24	0.8	1.25	100	17595	9751	5438	10.8	2.36
4.9	15.6	12	0.5	2	100	21297	5862	2977	10.8	4.14
4.9	15.6	24	0.5	2	100	22355	5247	2664	10.8	4.77
4.9	15.6	12	0.8	1.25	100	13663	9451	5271	10.8	2
4.9	15.6	24	0.8	1.25	100	14499	8673	4837	10.8	2.23
4.9	17.8	12	0.5	2	100	23290	6536	3319	10.8	4.07
4.9	17.8	24	0.5	2	100	24348	5921	3006	10.8	4.62
4.9	17.8	12	0.8	1.25	100	14908	10529	5872	10.8	1.97
4.9	17.8	24	0.8	1.25	100	15744	9751	5439	10.8	2.17
8.6	15.6	12	0.5	2	100	17155	5862	2977	10.8	3.44
8.6	15.6	24	0.5	2	100	18083	5247	2664	10.8	3.95
8.6	15.6	12	0.8	1.25	100	11031	9451	5271	10.8	1.73
8.6	15.6	24	0.8	1.25	100	11764	8673	4837	10.8	1.92
8.6	17.8	12	0.5	2	100	18689	6536	3318	10.8	3.37
8.6	17.8	24	0.5	2	100	19616	5921	3007	10.8	3.82
8.6	17.8	12	0.8	1.25	100	11989	10529	5873	10.8	1.7
8.6	17.8	24	0.8	1.25	100	12723	9751	5438	10.8	1.86

Table 9.2: Sliding analyses without struts or slab for low and high ranges of soil parameters based on reported investigation (continued)

Excav depth	γ_{soil}	**c**$_u$	**K**$_a$	**K**$_p$	**q**$_{sur}$	$\sum P_{passive}$	$\sum P_{active}$	R tan **δ**	**C**$_b$ **B**	**FOS**
11.8	15.6	12	0.5	2	100	13918	5862	2976	10.8	2.88
11.8	15.6	24	0.5	2	100	14733	5247	2664	10.8	3.32
11.8	15.6	12	0.8	1.25	100	8970	9451	5271	10.8	1.51
11.8	15.6	24	0.8	1.25	100	9615	8673	4837	10.8	1.67
11.8	17.8	12	0.5	2	100	15102	6536	3318	10.8	2.82
11.8	17.8	24	0.5	2	100	15917	5921	3006	10.8	3.20
11.8	17.8	12	0.8	1.25	100	9710	10529	5872	10.8	1.48
11.8	17.8	24	0.8	1.25	100	10355	9751	5438	10.8	1.62
15.2	15.6	12	0.5	2	100	10828	5862	2977	10.8	2.35
15.2	15.6	24	0.5	2	100	11524	5247	2664	10.8	2.7
15.2	15.6	12	0.8	1.25	100	6999	9451	5271	10.8	1.3
15.2	15.6	24	0.8	1.25	100	7549	8673	4837	10.8	1.43
15.2	17.8	12	0.5	2	100	11690	6536	3319	10.8	2.3
15.2	17.8	24	0.5	2	100	12386	5921	3006	10.8	2.6
15.2	17.8	12	0.8	1.25	100	7538	10529	5872	10.8	1.27
15.2	17.8	24	0.8	1.25	100	8088	9751	5439	10.8	1.39
17.3	15.6	12	0.5	2	100	9099	5862	2977	10.8	2.06
17.3	15.6	24	0.5	2	100	9721	5247	2664	10.8	2.36
17.3	15.6	12	0.8	1.25	100	5894	9451	5271	10.8	1.18
17.3	15.6	24	0.8	1.25	100	6386	8673	4837	10.8	1.3
17.3	17.8	12	0.5	2	100	9789	6536	3318	10.8	2.0
17.3	17.8	24	0.5	2	100	10411	5921	3007	10.8	2.27
17.3	17.8	12	0.8	1.25	100	6325	10529	5873	10.8	1.16
17.3	17.8	24	0.8	1.25	100	6817	9751	5438	10.8	1.26
19.7	15.6	12	0.5	2	100	7293	5862	2977	10.8	1.75
19.7	15.6	24	0.5	2	100	7830	5247	2664	10.8	2.0
19.7	15.6	12	0.8	1.25	100	4737	9451	5271	10.8	1.06
19.7	15.6	24	0.8	1.25	100	5162	8673	4837	10.8	1.15
19.7	17.8	12	0.5	2	100	7808	6536	3318	10.8	1.7
19.7	17.8	24	0.5	2	100	8346	5921	3007	10.8	1.92
19.7	17.8	12	0.8	1.25	100	5059	10529	5873	10.8	1.04
19.7	17.8	24	0.8	1.25	100	5484	9751	5438	10.8	1.12

Table 9.3: Summary of results of probability for sliding occurrence

Excavation stage	Excavation depth	FOS_{min}	FOS_{sure}	Failure occurrence probability	Result for sliding Failure occurrence probability
3	8.6	1.69	1.75	0.07	very low
4	11.8	1.48	1.75	0.35	Low to medium
5	15.2	1.27	1.75	0.63	Medium to high
6	17.3	1.16	1.75	0.79	High
7	19.7	1.04	1.75	0.95	Very high

Table 9.4: Possible internal damage and expected internal damage due sliding

Description of risk potential	**Probability of failure occurrence**	**Possible internal damage $ per meter (soil)**	**Possible internal damage $ per meter (RC demolish)**	**Possible internal damage $ per meter (RC reconstruct)**	**Possible internal damage $ per meter (early works)**
Sliding in 1st stage of excavation	0	0	0	0	0
Sliding in 2nd stage of excavation	0	0	0	0	0
Sliding in 3rd stage of excavation	0.07	129.5	1923	1741	1905
Sliding in 4th stage of excavation	0.35	243.7	2649	2390	2771
Sliding in 5th stage of excavation	0.63	404.3	3420	3078	3524
Sliding in 6th stage of excavation	0.79	523.8	3896	3503	3989
Sliding in 7th stage of excavation	0.95	679.2	4440	3989	4521

Table 9.4: Possible internal damage and expected internal damage due sliding (continued)

Description of risk potential	Possible internal damage $ per meter (overhead)	Possible internal damage $ per meter (equip)	Possible internal damage $ per meter	Expected internal damage $ per meter
Sliding in 1st stage of excavation	0	0	0	0
Sliding in 2nd stage of excavation	0	0	0	0
Sliding in 3rd stage of excavation	381	48	6128	430
Sliding in 4th stage of excavation	554	69	8676	3112
Sliding in 5th stage of excavation	705	88	11219	7110
Sliding in 6th stage of excavation	798	100	12810	10086
Sliding in 7th stage of excavation	904	113	14647	13885

Table 9.5: Possible external damages and expected external damage due sliding

Description of risk potential	Possible external damage $ per meter (road)	Possible external damage $ per meter (adjacent building)	Possible external damage $ per meter	Expected external damage $ per meter
Sliding in 1st stage of excavation	0	0	0	0
Sliding in 2nd stage of excavation	0	0	0	0
Sliding in 3rd stage of excavation	682.2	264	946.2	66.3
Sliding in 4th stage of excavation	928.6	3300	4228.6	1516.8
Sliding in 5th stage of excavation	1190.4	11000	12190.4	7725.6
Sliding in 6th stage of excavation	1352.1	30800	32152.1	25314.8
Sliding in 7th stage of excavation	1536.9	82500	84036.9	79667

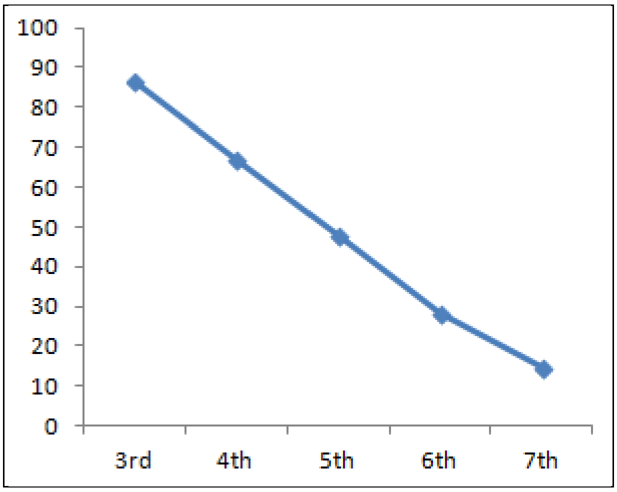

Figure 9.4: Internal damage shares (percent) in term of excavation stage for sliding

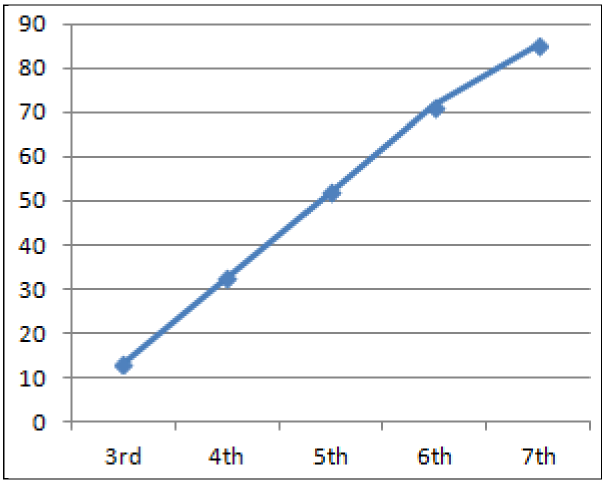

Figure 9.5: External damage shares (percent) in term of excavation stage for sliding

Table 9.6: Expected damages due sliding and its impact on project

Description of risk potential	**Expected internal damage $ per meter**	**Expected external damage $ per meter**	**Expected internal and external damage $ per meter**	**Expected accident $ per meter**	**Impact on project** (E)/(1800 ×13×550/ 192)
Sliding in 1st stage of excavation	0	0	0	2667.8× 0.5	1.99%
Sliding in 2nd stage of excavation	0	0	0	2667.8× 0.5	1.99%
Sliding in 3rd stage of excavation	430	66.3	496.3	2667.8× 0.5	2.73%
Sliding in 4th stage of excavation	3112.2	1516.8	4629	2667.8× 0.5	21.31%
Sliding in 5th stage of excavation	7110.1	7725.6	14835.7	2667.8× 0.5	63.76%
Sliding in 6th stage of excavation	10085.9	25314.8	35400.7	2667.8× 0.5	96.32%
Sliding in 7th stage of excavation	13885.4	79669	93554.4	2667.8× 0.5	137.12%

Table 9.7: Optimistic and Pessimistic impact due sliding after diaphragm wall construction

Description of risk potential	Optimistic expected damages and accident $	Pessimistic expected damages and accident $	Optimistic Impact on project	Average impact on project	Pessimistic Impact on project
Sliding in 1st stage of excavation	256109	256109	1.99%	1.99%	1.99%
Sliding in 2nd stage of excavation	256109	256109	1.99%	1.99%	1.99%
Sliding in 3rd stage of excavation	382633	410749	2.97%	3.08%	3.19%
Sliding in 4th stage of excavation	2670988	3207629	20.75%	22.81%	24.92%
Sliding in 5th stage of excavation	7675726	9324530	59.64%	66.01%	72.45%
Sliding in 6th stage of excavation	11517156	14019611	89.46%	99.21%	108.93%
Sliding in 7th stage of excavation	16320046	19889810	126.81%	140.71%	154.54%

Table 9.8: Resistant request by struts and slabs for increasing FOS of sliding up to 1.75 for slabs and up to 1.25 for steel struts

Second support level	Resist.	Action	Resist.	Resist.	F(kN)	A(cm^2)	Secondary support
-2.0					0	149	Steel strut
-3.5					0	0	Slab RC
-7.1	11989	10529	5872	10.8	553.95	1.32 st	Slab RC
-10.3	9710	10529	5872	10.8	2832.95	6.76 st	Slab RC
-13.7	7538	10529	5872	10.8	5004.95	11.92 st	Slab RC
-16.5	6325	10529	5872	10.8	953.45	198	Steel strut
-17.1	5059	10529	5872	10.8	7483.95	17.82	Slab RC

Table 9.9: Overturning analyses for low and high ranges of soil parameters based on reported investigation

Excavation depth	γ_{soil}	c_u	K_a	K_p	q_{sur}	$\sum M_o$	$\sum M_R$	**FOS**
2.8	15.6	12	0.5	2	100	12492	5754	0.4606
2.8	15.6	24	0.5	2	100	12492	5754	0.4606
2.8	15.6	12	0.8	1.25	100	15471	3732	0.2412
2.8	15.6	24	0.8	1.25	100	15471	3732	0.2412
2.8	17.8	12	0.5	2	100	12716	6514	0.5123
2.8	17.8	24	0.5	2	100	12716	6514	0.5123
2.8	17.8	12	0.8	1.25	100	15830	4207	0.2658
2.8	17.8	24	0.8	1.25	100	15830	4207	0.2658
4.9	15.6	12	0.5	2	100	13664	5074	0.3713
4.9	15.6	24	0.5	2	100	13664	5074	0.3713
4.9	15.6	12	0.8	1.25	100	17624	3307	0.1876
4.9	15.6	24	0.8	1.25	100	17624	3307	0.1876
4.9	17.8	12	0.5	2	100	13889	5738	0.4131
4.9	17.8	24	0.5	2	100	13889	5738	0.4131
4.9	17.8	12	0.8	1.25	100	17984	3722	0.207
4.9	17.8	24	0.8	1.25	100	17984	3722	0.207
8.6	15.6	12	0.5	2	100	15497	3986	0.2572
8.6	15.6	24	0.5	2	100	15497	3986	0.2572
8.6	15.6	12	0.8	1.25	100	21025	2627	0.125
8.6	15.6	24	0.8	1.25	100	21025	2627	0.125
8.6	17.8	12	0.5	2	100	15722	4498	0.2861
8.6	17.8	24	0.5	2	100	15722	4498	0.2861
8.6	17.8	12	0.8	1.25	100	21385	2947	0.1378
8.6	17.8	24	0.8	1.25	100	21385	2947	0.1378
11.8	15.6	12	0.5	2	100	16843	3161	0.1877
11.8	15.6	24	0.5	2	100	16843	3161	0.1877
11.8	15.6	12	0.8	1.25	100	23562	2112	0.0896
11.8	15.6	24	0.8	1.25	100	23562	2112	0.0896
11.8	17.8	12	0.5	2	100	17068	3556	0.2083
11.8	17.8	24	0.5	2	100	17068	3556	0.2083
11.8	17.8	12	0.8	1.25	100	23921	2358	0.0986
11.8	17.8	24	0.8	1.25	100	23921	2358	0.0986

Table 9.9: Overturning analyses for low and high ranges of soil parameters based on reported investigation (continued)

Excavation depth	γ_{soil}	**c_u**	**K_a**	**K_p**	**q_{sur}**	∑Mo	$\sum M_R$	**FOS**
15.2	15.6	12	0.5	2	100	18030	2401	0.1332
15.2	15.6	24	0.5	2	100	18030	2401	0.1332
15.2	15.6	12	0.8	1.25	100	25846	1636	0.0633
15.2	15.6	24	0.8	1.25	100	25846	1636	0.0633
15.2	17.8	12	0.5	2	100	18255	2688	0.1473
15.2	17.8	24	0.5	2	100	18255	2688	0.1473
15.2	17.8	12	0.8	1.25	100	26205	1816	0.0693
15.2	17.8	24	0.8	1.25	100	26205	1816	0.0693
17.3	15.6	12	0.5	2	100	18639	1991	0.1068
17.3	15.6	24	0.5	2	100	18639	1991	0.1068
17.3	15.6	12	0.8	1.25	100	27044	1380	0.051
17.3	15.6	24	0.8	1.25	100	27044	1380	0.051
17.3	17.8	12	0.5	2	100	18863	2221	0.1177
17.3	17.8	24	0.5	2	100	18863	2221	0.1177
17.3	17.8	12	0.8	1.25	100	27404	1524	0.0556
17.3	17.8	24	0.8	1.25	100	27404	1524	0.0556

Table 9.9: Overturning analyses for low and high ranges of soil parameters based on reported investigation (continued)

Excavation depth	γ_{soil}	**c_u**	**K_a**	**K_p**	**q_{sur}**	$\sum Mo$	$\sum M_R$	**FOS**
19.7	15.6	12	0.5	2	100	19217	1580	0.082
19.7	15.6	24	0.5	2	100	19217	1580	0.082
19.7	15.6	12	0.8	1.25	100	28216	1123	0.04
19.7	15.6	24	0.8	1.25	100	28216	1123	0.04
19.7	17.8	12	0.5	2	100	19441	1751	0.09
19.7	17.8	24	0.5	2	100	19441	1751	0.09
19.7	17.8	12	0.8	1.25	100	28576	1230	0.043
19.7	17.8	24	0.8	1.25	100	28576	1230	0.043

Table 9.10: Summary of results of probability occurrence for overturning failure

Excavation stage	**Excavation depth**	**FOS_{min}**	**FOS_{sure}**	**Failure occurrence probability**	**Result for overturning Failure occurrence probability**
1	2.8	0.24123	2	1	Certainly
2	4.9	0.18762	2	1	Certainly
3	8.6	0.12496	2	1	Certainly
4	11.8	0.08962	2	1	Certainly
5	15.2	0.06331	2	1	Certainly
6	17.3	0.05104	2	1	Certainly
7	19.7	0.0398	2	1	Certainly

Table 9.11: Resistant request by struts and slabs for increasing FOS_{min} of overturning up to 2 for slabs and up to 1.25 for steel struts

Second support level	M_O	M_r	M_{add} **(kNm/m)**	**F (kN/m)**	**A(cm²)**	**Secondary support**
-2.0	15471	3732	15607	882	4.79/m (149)	Steel strut
-3.5	17624	3306	31942	1972	8.58 st	Slab (RC)
-7.1	21025	2627	32621	2589	11.26 st	Slab (RC)
-10.3	23562	2111	45013	4789	20.82 st	Slab (RC)
-13.7	25846	1636	50056	8343	36.27 st	Slab (RC)
-16.5	27044	1380	52708	16472	68.68 st	Slab (RC)
-17.1	28216	1123	34147	13134	71/m (198)	Steel strut

Table 9.12: Basel heave analysis for low and high ranges of soil parameters based on reported investigation (FOS ≥ 1.2)

Wall length	**Excavation depth**	γ_{soil}	c_u	$q_{surcharge}$	**r**	**W**	**FOS**
35	2.8	15.6	12	100	34.2	1406.496	0.734
35	2.8	15.6	24	100	34.2	1406.496	1.05
35	2.8	17.8	12	100	34.2	1604.848	0.503
35	2.8	17.8	24	100	34.2	1604.848	2.855
35	4.9	15.6	12	100	31.5	2407.86	0.73
35	4.9	15.6	24	100	31.5	2407.86	1.459
35	4.9	17.8	12	100	31.5	2747.43	13.23
35	4.9	17.8	24	100	31.5	2747.43	1.449
35	8.6	15.6	12	100	31.5	4226.04	0.732
35	8.6	15.6	24	100	31.5	4226.04	1.465
35	8.6	17.8	12	100	31.5	4822.02	6.958
35	8.6	17.8	24	100	31.5	4822.02	1.446
35	11.8	15.6	12	100	27.9	5135.832	5.101
35	11.8	15.6	24	100	27.9	5135.832	1.397
35	11.8	17.8	12	100	27.9	5860.116	4.471
35	11.8	17.8	24	100	27.9	5860.116	1.371
35	15.2	15.6	12	100	24.7	5856.864	3.431
35	15.2	15.6	24	100	24.7	5856.864	1.326
35	15.2	17.8	12	100	24.7	6682.832	0.645
35	15.2	17.8	24	100	24.7	6682.832	1.291
35	17.3	15.6	12	100	21.3	5748.444	2.655
35	17.3	15.6	24	100	21.3	5748.444	1.241
35	17.3	17.8	12	100	21.3	6559.122	0.601
35	17.3	17.8	24	100	21.3	6559.122	4.654
35	19.7	15.6	12	100	18.5	5685.42	0.579
35	19.7	15.6	24	100	18.5	5685.42	1.158
35	19.7	17.8	12	100	18.5	6487.21	0.556
35	19.7	17.8	24	100	18.5	6487.21	1.113

Table 9.13: The result of surcharge reducing for Basal heave eliminating

Wall length	**Excavation depth**	γ_{soil}	c_u	$q_{surcharge}$	**r**	**W**	**FOS**
35	2.8	15.6	12	13	34.2	1406.496	4.798
35	2.8	15.6	24	13	34.2	1406.496	2.660
35	2.8	17.8	12	13	34.2	1604.848	1.20
35	2.8	17.8	24	13	34.2	1604.848	6.809
35	4.9	15.6	12	13	31.5	2407.86	4.241
35	4.9	15.6	24	13	31.5	2407.86	8.482
35	4.9	17.8	12	13	31.5	2747.43	13.23
35	4.9	17.8	24	13	31.5	2747.43	8.159
35	8.6	15.6	12	13	31.5	4226.04	3.484
35	8.6	15.6	24	13	31.5	4226.04	6.967
35	8.6	17.8	12	13	31.5	4822.02	6.958
35	8.6	17.8	24	13	31.5	4822.02	6.561
35	11.8	15.6	12	13	27.9	5135.832	5.101
35	11.8	15.6	24	13	27.9	5135.832	5.608
35	11.8	17.8	12	13	27.9	5860.116	4.471
35	11.8	17.8	24	13	27.9	5860.116	5.205
35	15.2	15.6	12	13	24.7	5856.864	3.431
35	15.2	15.6	24	13	24.7	5856.864	4.448
35	15.2	17.8	12	13	24.7	6682.832	2.038
35	15.2	17.8	24	13	24.7	6682.832	4.075
35	17.3	15.6	12	13	21.3	5748.444	2.655
35	17.3	15.6	24	13	21.3	5748.444	3.723
35	17.3	17.8	12	13	21.3	6559.122	1.694
35	17.3	17.8	24	13	21.3	6559.122	4.654
35	19.7	15.6	12	13	18.5	5685.42	1.526
35	19.7	15.6	24	13	18.5	5685.42	3.052
35	19.7	17.8	12	13	18.5	6487.21	1.379
35	19.7	17.8	24	13	18.5	6487.21	2.758

Table 9.14: Bottom heave due unloading analysis for low and high ranges of soil parameters based on reported investigation

Wall length	**Excav depth**	γ_{soil}	c_u	$q_{surcharge}$	**Nc** (H>B)	**Y**	**Nc** (H<B)	**FOS** (H>B)	**FOS** (H<B)
35	2.8	15.6	12	100	6	30.2	7	0.501	1.973
35	2.8	15.6	24	100	6	30.2	7	1.002	4.053
35	2.8	15.6	12	100	7	30.2	8	0.585	2.255
35	2.8	15.6	24	100	7	30.2	8	1.169	4.632
35	2.8	17.8	12	100	6	30.2	7	0.481	1.724
35	2.8	17.8	24	100	6	30.2	7	0.961	3.528
35	2.8	17.8	12	100	7	30.2	8	0.561	1.970
35	2.8	17.8	24	100	7	30.2	8	1.121	4.032
35	4.9	15.6	12	100	6	28.1	7	0.408	1.130
35	4.9	15.6	24	100	6	28.1	7	0.816	2.325
35	4.9	15.6	12	100	7	28.1	8	0.476	1.291
35	4.9	15.6	24	100	7	28.1	8	0.952	2.657
35	4.9	17.8	12	100	6	28.1	7	0.386	0.987
35	4.9	17.8	24	100	6	28.1	7	0.769	2.023
35	4.9	17.8	12	100	7	28.1	8	0.449	1.128
35	4.9	17.8	24	100	7	28.1	8	0.897	2.312
35	8.6	15.6	12	100	6	24.4	7	0.307	0.646
35	8.6	15.6	24	100	6	24.4	7	0.615	1.337
35	8.6	15.6	12	100	7	24.4	8	0.359	0.739
35	8.6	15.6	24	100	7	24.4	8	0.717	1.527
35	8.6	17.8	12	100	6	24.4	7	0.284	0.564
35	8.6	17.8	24	100	6	24.4	7	0.569	1.162
35	8.6	17.8	12	100	7	24.4	8	0.332	0.645
35	8.6	17.8	24	100	7	24.4	8	0.664	1.328
35	11.8	15.6	12	100	6	21.2	7	0.253	0.474
35	11.8	15.6	24	100	6	21.2	7	0.507	0.984
35	11.8	15.6	12	100	7	21.2	8	0.296	0.541
35	11.8	15.6	24	100	7	21.2	8	0.591	1.125
35	11.8	17.8	12	100	6	21.2	7	0.232	0.413
35	11.8	17.8	24	100	6	21.2	7	0.464	0.854
35	11.8	17.8	12	100	7	21.2	8	0.271	0.472
35	11.8	17.8	24	100	7	21.2	8	0.542	0.976

Table 9.14: Bottom heave analysis due unloading for low and high ranges of soil parameters based on reported investigation (continued)

Wall length	**Excav depth**	γ_{soil}	c_u	$q_{surcharge}$	**Nc** (H>B)	**Y**	**Nc** (H<B)	**FOS** (H>B)	**FOS** (H<B)
35	15.2	15.6	12	100	6	17.8	7	0.214	0.370
35	15.2	15.6	24	100	6	17.8	7	0.427	0.776
35	15.2	15.6	12	100	7	17.8	8	0.249	0.423
35	15.2	15.6	24	100	7	17.8	8	0.498	0.886
35	15.2	17.8	12	100	6	17.8	7	0.194	0.323
35	15.2	17.8	24	100	6	17.8	7	0.389	0.672
35	15.2	17.8	12	100	7	17.8	8	0.227	0.369
35	15.2	17.8	24	100	7	17.8	8	0.453	0.768
35	17.3	15.6	12	100	6	15.7	7	0.195	0.327
35	17.3	15.6	24	100	6	15.7	7	0.389	0.690
35	17.3	15.6	12	100	7	15.7	8	0.227	0.374
35	17.3	15.6	24	100	7	15.7	8	0.454	0.789
35	17.3	17.8	12	100	6	15.7	7	0.176	0.285
35	17.3	17.8	24	100	6	15.7	7	0.353	0.597
35	17.3	17.8	12	100	7	15.7	8	0.206	0.326
35	17.3	17.8	24	100	7	15.7	8	0.412	0.682
35	19.7	15.6	12	100	6	13.3	7	0.177	0.290
35	19.7	15.6	24	100	6	13.3	7	0.354	0.618
35	19.7	15.6	12	100	7	13.3	8	0.206	0.332
35	19.7	15.6	24	100	7	13.3	8	0.412	0.706
35	19.7	17.8	12	100	6	13.3	7	0.160	0.252
35	19.7	17.8	24	100	6	13.3	7	0.320	0.533
35	19.7	17.8	12	100	7	13.3	8	0.186	0.288
35	19.7	17.8	24	100	7	13.3	8	0.373	0.609

Table 9.15: Heaven due artesian pressure for low and high ranges of soil parameters based on reported investigation

Wall length	**Excavation depth**	γ_{soil}	γ_w	**w**	**u**	**FOS**
35	2.8	15.6	10	502.32	330	1.522
35	2.8	17.8	10	573.16	330	1.737
35	4.9	15.6	10	469.56	330	1.423
35	4.9	17.8	10	535.78	330	1.624
35	8.6	15.6	10	411.84	330	1.248
35	8.6	17.8	10	469.92	330	1.424
35	11.8	15.6	10	361.92	330	1.097
35	11.8	17.8	10	412.96	330	1.251
35	15.2	15.6	10	308.88	330	0.936
35	15.2	17.8	10	352.44	330	1.068
35	17.3	15.6	10	276.12	330	0.837
35	17.3	17.8	10	315.06	330	0.955
35	19.7	15.6	10	238.68	330	0.723
35	19.7	17.8	10	272.34	330	0.825

Table 9.16: Summary of results of probability occurrence of artesian heaven failure

Excav stage	**Excav depth**	**FOS**$_{min}$	**FOS**$_{sure}$	**Failure occurrence probability**	**Result for artesian heavan Failure occurrence probability**
1	2.8	1.522	1.2	0	Impossible
2	4.9	1.423	1.2	0	Impossible
3	8.6	1.248	1.2	0	Impossible
4	11.8	1.097	1.2	0.515	Medium
5	15.2	0.936	1.2	1	Certainly
6	17.3	0.837	1.2	1	Certainly
7	19.7	0.723	2	1	Certainly

Table 9.17: Hydraulic failure analyzing based on Terzaghi method for low and high ranges of soil parameters based on reported investigation

Wall length	**Excavation depth**	γ_{soil}	γ_W	γ'	L_d	h_w	**FOS**
35	2.8	15.6	10	5.6	32.2	0.8	45.08
35	2.8	17.8	10	7.8	32.2	0.8	62.79
35	4.9	15.6	10	5.6	30.1	2.9	11.625
35	4.9	17.8	10	7.8	30.1	2.9	16.192
35	8.6	15.6	10	5.6	26.4	6.6	4.48
35	8.6	17.8	10	7.8	26.4	6.6	6.24
35	11.8	15.6	10	5.6	23.2	9.8	2.651
35	11.8	17.8	10	7.8	23.2	9.8	3.693
35	15.2	15.6	10	5.6	19.8	13.2	1.68
35	15.2	17.8	10	7.8	19.8	13.2	2.34
35	17.3	15.6	10	5.6	17.7	15.3	1.296
35	17.3	17.8	10	7.8	17.7	15.3	1.805
35	19.7	15.6	10	5.6	15.3	17.7	0.968
35	19.7	17.8	10	7.8	15.3	17.7	1.348

Table 9.18: Hydraulic failure analyzing based on critical hydraulic gradient method for low and high ranges of soil parameters based on reported investigation

Excavation depth	γ_{soil}	γ'	L_0	L_1	L_2	FOS_0	FOS_1	FOS_2
2.8	15.6	5.6	83	82	81	2.63	2.75	2.89
2.8	17.8	7.8	83	82	81	2.66	3.83	4.02
4.9	15.6	5.6	78.8	77.8	76.8	2.49	2.61	2.74
4.9	17.8	7.8	78.8	77.8	76.8	3.47	3.63	3.82
8.6	15.6	5.6	71.4	70.4	69.4	2.26	2.36	2.48
8.6	17.8	7.8	71.4	70.4	69.4	3.15	3.29	3.45
11.8	15.6	5.6	65	64	63	2.06	2.15	2.25
11.8	17.8	7.8	65	64	63	2.86	2.99	3.13
15.2	15.6	5.6	58.2	57.2	56.2	1.84	1.92	2
15.2	17.8	7.8	58.2	57.2	56.2	2.57	2.67	2.79
17.3	15.6	5.6	54	53	52	1.71	1.78	1.85
17.3	17.8	7.8	54	53	52	2.38	2.48	2.58
19.7	15.6	5.6	49.2	48.2	47.2	1.56	1.62	1.68
19.7	17.8	7.8	49.2	48.2	47.2	2.17	2.25	2.34

Table 9.18: Hydraulic failure analyzing based on critical hydraulic gradient method for low and high ranges of soil parameters based on reported investigation (continue)

Excavation depth	γ_{soil}	γ'	**L**$_3$	**L**$_{15.7}$	**FOS**$_3$	**FOS**$_4$
2.8	15.6	5.6	80	67.3	3.05	18.8
2.8	17.8	7.8	80	67.3	4.24	26.2
4.9	15.6	5.6	75.8	63.1	2.89	17.7
4.9	17.8	7.8	75.8	63.1	4.02	24.6
8.6	15.6	5.6	68.4	55.7	2.61	15.6
8.6	17.8	7.8	68.4	55.7	3.63	21.7
11.8	15.6	5.6	62	49.3	2.36	13.8
11.8	17.8	7.8	62	49.3	3.29	19.2
15.2	15.6	5.6	55.2	42.5	2.1	11.9
15.2	17.8	7.8	55.2	42.5	2.93	16.6
17.3	15.6	5.6	51	38.3	1.94	10.7
17.3	17.8	7.8	51	38.3	2.71	14.9
19.7	15.6	5.6	46.2	33.5	1.76	9.38
19.7	17.8	7.8	46.2	33.5	2.45	13.1

9.6.3 Geotechnical risk response plan

After risk assessment, let us prepare geotechnical risk response plan for risk management. Based on data acquired by early risk analysis, the geotechnical risk response plan for project can be prepare as shown in table 9.19.

Table 9.19 Geotechnical risk response plan

Description of risk potential	Sliding	Overturning	Basel heaves
Risk in 1st stage of excavation	Deal with 22-23 kg/m^2 steel struts otherwise discourage the project	Deal with 22-23 kg/m^2 steel struts otherwise discourage the project	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept it and mitigate it by excavation
Risk in 2nd stage of excavation	Deal with permanent RC slabs	Deal with permanent RC slabs	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept and mitigate it by excavation
Risk in 3rd stage of excavation	Deal with permanent RC slabs	Deal with permanent RC slabs	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept and mitigate it by excavation
Risk in 4th stage of excavation	Deal with permanent RC slabs	Deal with permanent RC slabs	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept and mitigate it by excavation
Risk in 5th stage of excavation	Deal with permanent RC slabs	Deal with permanent RC slabs	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept and mitigate it by excavation
Risk in 6th stage of excavation	Deal with permanent RC slabs	Deal with permanent RC slabs	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept and mitigate it by excavation
Risk in 7th stage of excavation	Deal with 26-27 kg/m^2 steel struts	Deal with 26-27 kg/m^2 steel struts	Eliminate by decreasing surcharge to 13kN/m^2 otherwise accept and mitigate it by excavation

Table 9.19 Geotechnical risk response plan (continued)

Description of risk potential	**Bottom heave due unloading**	**Heaven due artesian pressure**	**Hydraulic failure**
Risk in 1st stage of excavation	Accept it and mitigate it by excavation	There isn't	There isn't
Risk in 2nd stage of excavation	Accept it and mitigate it by excavation	There isn't	There isn't
Risk in 3rd stage of excavation	Accept it and mitigate it by excavation	There isn't	There isn't
Risk in 4th stage of excavation	Accept it and mitigate it by excavation	Mitigate by dewatering before and during excavation otherwise discourage the project	There isn't
Risk in 5th stage of excavation	Accept it and mitigate it by excavation	Mitigate by dewatering before and during excavation otherwise discourage the project	There isn't
Risk in 6th stage of excavation	Accept it and mitigate it by excavation	Mitigate by dewatering before and during excavation otherwise discourage the project	There isn't
Risk in 7th stage of excavation	Accept it and mitigate it by excavation	Mitigate by dewatering before and during excavation otherwise discourage the project	Mitigate by dewatering before and during excavation otherwise discourage the project

Table 9.19 Geotechnical risk response plan (continued)

Description of risk potential	**Dewatering**	**Upheaval failure**	**Liquefaction or sand boiling failure**
Risk in 1st stage of excavation	Not required, avoid it	There isn't	There isn't
Risk in 2nd stage of excavation	Not required, avoid it	There isn't	There isn't
Risk in 3rd stage of excavation	Not required, avoid it	There isn't	There isn't
Risk in 4th stage of excavation	Deal with adjacent properties damages due settlement otherwise discourage the project	There isn't	There isn't
Risk in 5th stage of excavation	Deal with adjacent properties damages due settlement otherwise discourage the project	There isn't	There isn't
Risk in 6th stage of excavation	Deal with adjacent properties damages due settlement otherwise discourage the project	There isn't	There isn't
Risk in 7th stage of excavation	Deal with adjacent properties damages due settlement otherwise discourage the project	There isn't	There isn't

9.7 Multi-level anchored contiguous pile wall in Northern Cyprus

The project is named Spring Mall in Ibrahim Pasha Quarter in Lefkosha (Nicosia), North Cyprus. The land area is 7500 m^2. Designed building area in surface is 6550 m^2 and total building area is 50750 m^2 (5 level underground + ground level + 7 story) that was planned in 2015. The initial land plan is shown in figure 9.6. Based on plan the maximum depth of excavation was 20 meters.

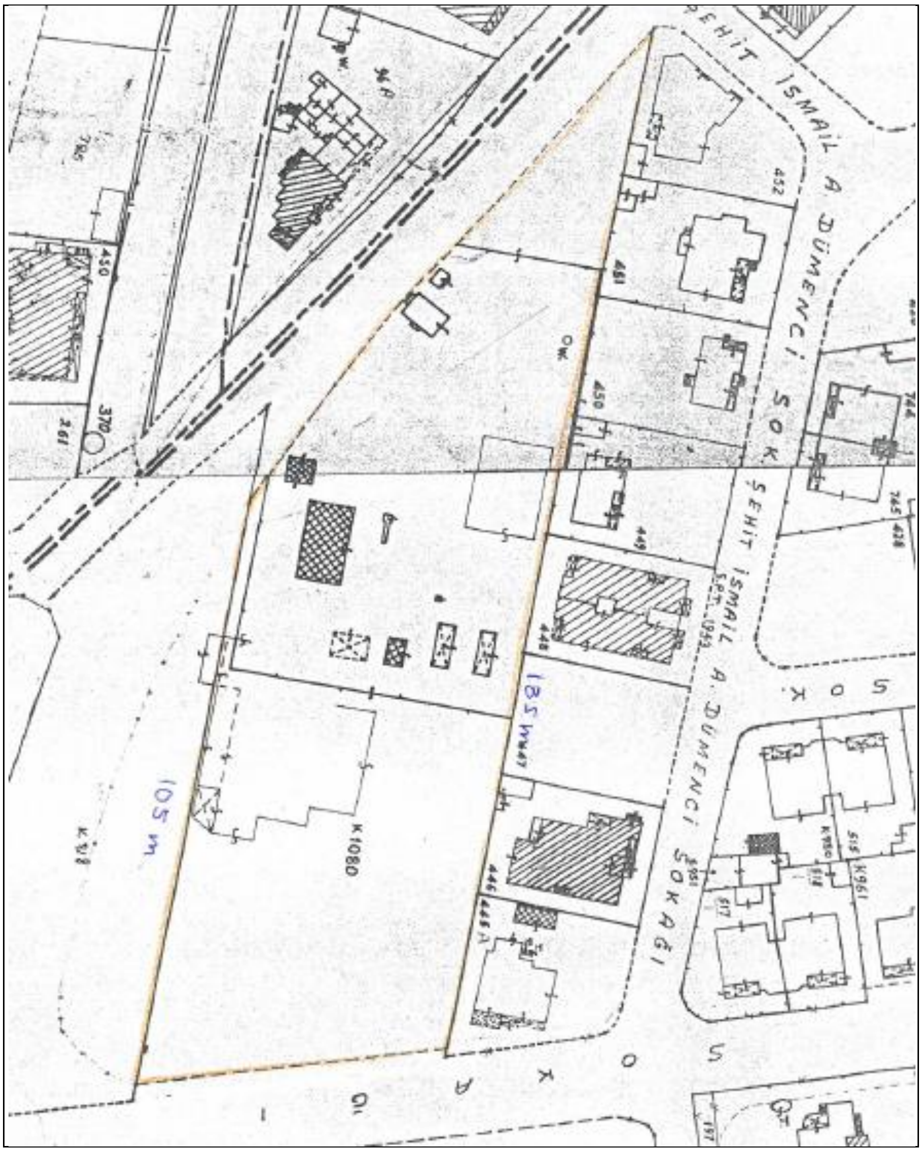

Figure 9.6: Spring Mall Initial land plans and adjacent lands and properties

9.7.1The site geotechnical investigation

The site geotechnical investigation was done by four boreholes on 12/2014 that the important test results are indicated in table 9.20 [104]. Fifty grading tests were done in different depths of four boreholes. Summary of SPT-N_{30} test results and important plastic properties are shown in tables 9.21 and 9.22. In depths between -18.0 to -20.0 the range of SPT-N_{30} is 10-45. Also q_{allow} = 3.12 kg/cm^2, N=23, q_{net} =2 kg/cm^2 , ΔH= 2.71 cm < 7.5 cm [104].

Table 9.20: The site geotechnical important samples test results [104]

Depth : (level -meter)	**BH: SK2**	**BH:SK3**	**BH:SK4**	**BH:SK6**
0.0 to -3.0	intermediate	intermediate	intermediate	intermediate
-3.0 to -3.5 SK3:Sandy clay	intermediate	LL=33.1 PL=20.8 w=15	intermediate	intermediate
-3.5 to -14.0	intermediate	intermediate	intermediate	intermediate
-14.0 to -14.5 SK2: Clay	LL=39.3 PL=23.4 w=20.2	intermediate	intermediate	intermediate
-14.5 to -15.5	intermediate	intermediate	intermediate	intermediate
-15.5 to -16.0 SK6: Clay	intermediate	intermediate	intermediate	LL=30.7 PL=19.3 w=19.2
-15.5 to -16.5	intermediate	intermediate	intermediate	intermediate
-16.5 to -17.0 SK4: Clay	intermediate	intermediate	w=15 γ=1.788gr/cm^3 ϕ =17.82 c = 61.76kPa	intermediate
-17.0 to -19.5	intermediate	intermediate	intermediate	intermediate
-19.5 to -20.0 SK6:Clay	intermediate	intermediate	intermediate	w=32.3 γ=1.716gr/cm^3 c=48.43kPa q_u=96.87kPa
-20 to -20.5	intermediate	intermediate	intermediate	intermediate
-20.5 to -21.0 SK3: Clay	intermediate	LL=55.8 PL=22.5 w=26.6	intermediate	intermediate
-21.0 to -24.95	intermediate	intermediate	intermediate	intermediate
-22 to -22.40	water table	water table	water table	water table
-22.40to-24.95	intermediate	intermediate	intermediate	Intermediate

Intermediate soils are sandy clay(S/C) or gravely clays(G/C) with gravel (S/C+G)or with sand (G/C+S)

Table 9.21: The standard penetration test result (N_{30}) into different depths of four boreholes samples and corresponding soil [104]

Depth (m)	**SK2** N_{30}	**SK3** N_{30}	**SK4** N_{30}	**SK6** N_{30}	**SK2**	**SK3**	**SK4**	**SK6**
1.5	7	10	45	69	S/C+ G			C/S/G
3.0	10	8	17	32	S/C+ G	S/C+ G	C/S/G	C/S/G
4.5	57	49		57	C/S/G	C/S/G	C/S/G	C/S/G
6.0	47	18	32	25	C/S/G			C/S+G
7.5		7			C/S/G	S/C+ G		
8.0				11				S/C+G
9.0	9	10	33	11		S/C+ G		S/C+G
10.5	13	47	18		S/C+ G	G/C+S	G/C+S	C/S/G
12.0	74	18			C/S/G	S/C+ G		C/S/G
14.5	18				C/S/G	S/C+ G		
15.0		37	20	20		C/S/G		S/C+G
16.0	23			21				
16.5		20			S/C+ G	S/C+G		S/C+G
17.0			18				S/C+G	
18.0	18	18		21	S/C+ G	G/C+S		S/C+G
19.0			10				S/C+G	
19.5		33				S/C+G		
20.0	45				S/C+ G			
21.0			22					
21.50	36							
22.50		50	33	65		S/C+G		S/C+G
24.5	42	35		73		S/C+G		S/C+G
25.5							C/S/G	

Table 9.22: The site investigation crucial plasticity and shear strength parameters

Bh	**Depth** (m)	**LL** (%)	**PL** (%)	**PI** (%)	**W** (%)	**γ** (g/cm^3)	**ϕ** (^{o})	**c** (kPa)	$\mathbf{q_u}$ (kPa)	$\mathbf{I_c}$
SK2	14-14.5	39.3	23.4	15.9	20.2					1.2
SK3	3-3.5	33.1	20.8	12.3	15					1.47
SK3	20.5-21	55.8	22.5	33.5	26.6					0.87
SK4	16.5-17				34.2	1.788	17.82	61.76		
SK6	15.5-16	30.7	19.3	11.4	19.2					1.0
SK6	19.5-20				32.3	1.716			96.87	

$PI = LL - PL$
$I_c = (LL – w) / PI$
$q_u = c \times N_c + \gamma_{soil} \times D_f \times N_q$
$N_q = e^{\pi \times \tan\phi'} \times \tan^2 [45^o + \phi'/2]$
$N_c = (N_q – 1) / \tan\phi$
$q_a = q_u/3$

9.7.2 Selected design alternative

Selective alternative is [104] Contiguous piles wall (1.5 m penetration) plus six level anchors (7 stages excavation, 2 m horizontal spacing) which cross-section is shown in figure 9.7 [109].

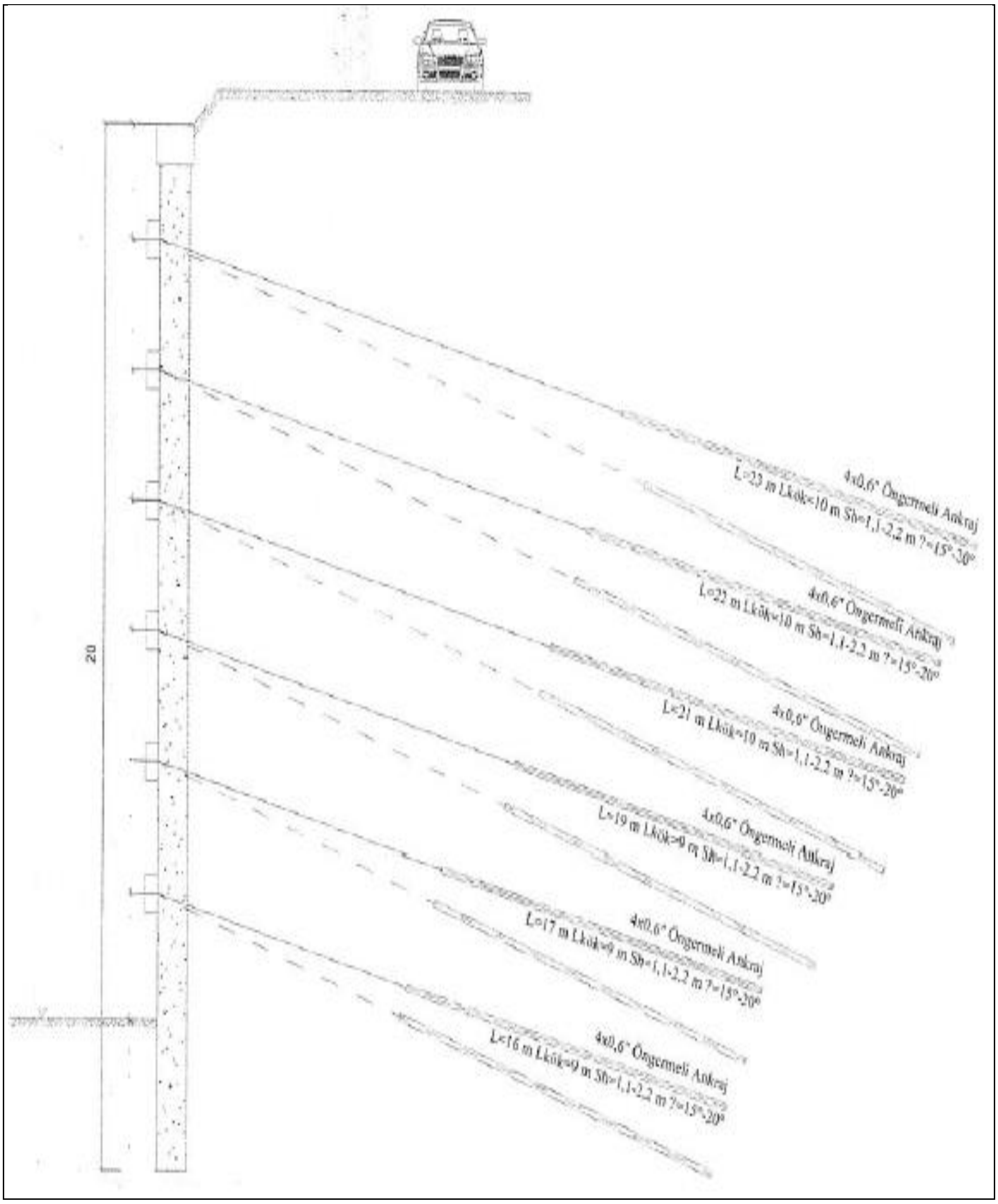

Figure 9.7: Cross-section of retaining wall and multi level anchors [109]

Design of each contiguous pile length and diameter are 21.5 m, and 0.85m respectively [109]. The steel longitudinal reinforcements are 19ϕ22 for each contiguous pile [109]. The steel shear reinforcement for each contiguous pile is ϕ10 at 15 cm [109]. The concrete class for each contiguous pile is C35 [109]. Number of contiguous pile in perimeter of building plan are 358 (0.2 m distance) [109]. The upper beam which is the top of the bored piles has 1.0×0.8 m, longitudinal reinforcement 6ϕ22 (top-bot) +2ϕ22bot, shear reinforcement ϕ10 at 15 cm, and concrete C35 [109].

Anchorages of 5ϕ18 (4ϕ22 performed) with horizontal spacing about 2.0 meters in each excavation stage, 23 meters length on first stage, 22 m length on second stage, 21 m length on third stage,19 m length on fourth stage, 17 m length on fifth stage, and 16 m length on sixth stage for each bar is planned [109]. Anchors support beam at the side of piles has 0.4×0.8 m cross-section, 5ϕ18 longitudinal reinforcements, and ϕ12at12cm shear reinforcements [109]. Excavation stages levels and volume of each stage vs. soil type and anchorage stages levels are indicated in table 9.23 [104, 109].

Table 9.23: Excavation and anchorage stages levels and volume of excavation in each stage vs. soil type

Excavation, **stage level to level**	**Anchor level**	**Excavation volume** (Bm^3)	**Soil type**
Pre: 0.15, -1.0		7550	S/C+ G , C/S/G
1: -1.0, -3.3	-3.0	15065	S/C+G, C/S/G
2: -3.3, -6.3	-6.0	19650	S/C+G, C/S/G
3: -6.3, -9.3	-9.0	19650	C/S/G, S/C+G, C/S+G
4: -9.3, -12.3	-12.0	19650	S/C+G, G/C+S, C/S/G
5: -12.3, -15.3	-15.0	19650	C/S/G, S/C+G
6: -15.3, -18.3	-18.0	19650	S/C+G, G/C+S
7: -18.3, -20.0		11135	S/C+G

S/C+ G = sandy clay with gravel, C/S/G = clayey sandy gravel, C/S+G = clayey sand with gravel, G/C+S = gravely clay with sand, S/C+G = sandy clay with gravel

9.7.3Review of activities definition and related quantities

Main activities as sequentially are: pre-excavation to -1 level, marking the location of wells, drilling the wells for contiguous bored piles, reinforcement and install the steel cages, Concreting the wells of bored piles (C35), reinforcement and formwork and concreting the upper beam on top of bored piles, excavation in seven stages , support beam hug the piles (for anchorage) including formwork and reinforcement and concreting jointly plate installing into in after each excavation stages, horizontal drilling and anchor installing and grouting on support beam hug the piles, final grading. Work breakdown structure [64, 114] is prepared which is shown in table 9.23.

Table 9.24: Work breakdown structure for Spring Mall deep excavation

ID	**WBS**	**Task**	**Definition, quantity**
1	1	Deep excavation activities	
2	1.1	Pre-excavation to -1 level	7550 Bm^3, surface area =6550 m^2
3	1.2	Contiguous pile wall	
4	1.2.1	Marking the location of wells for piles	358 pile location, diameter = 0.85, distance = 0.20 m
5	1.2.2	Drilling the wells for bored piles	4165 Bm^3, (358×)
6	1.2.3	Reinforcement and install the steel cages	549954 kg
7	1.2.4	Concreting the wells of bored piles	4165 m^3
8	1.2.5	Reinforcement the upper beam on bored piles	14450 kg
9	1.2.6	Formwork the upper beam on bored piles	718 m^2
10	1.2.7	Concreting the upper beam on bored piles	290.4 m^3
11	1.3	Excavation stage 1	15065 Bm^3 (bottom level= -3.3)
12	1.4	Anchorage stage 1	
13	1.4.1	Support beam Reinforcement	7725 kg
14	1.4.2	Support beam Formwork	436 m^2
15	1.4.3	Support beam Concreting	C25:116 m^3

Table 9.24: Work breakdown structure for Spring Mall deep excavation (continued)

ID	WBS	Task	Definition, quantity
16	1.4.4	Horizontal drilling	238 at 1.5 m, L=23 m
17	1.4.5	Anchor install and grouting	Anchor =5467 m (54473 kg), ,grouting= 4 m
18	1.5	Excavation stage 2	19650 Bm^3 (bottom level= -6.3)
19	1.6	Anchorage stage 2	
20	1.6.1	Support beam Reinforcement	7725 kg
21	1.6.2	Support beam Formwork	436 m^2
22	1.6.3	Support beam Concreting	C25:116 m^3
23	1.6.4	Horizontal drilling	238 at 1.5 m, L=22m
24	1.6.5	Anchor install and grouting	Anchor = 5228.5 m (52092 kg), grouting= 4×238 m
25	1.7	Excavation stage 3	19650 Bm^3 (bottom level= -9.3)
26	1.8	Anchorage stage 3	
27	1.8.1	Support beam Reinforcement	7725 kg
28	1.8.2	Support beam Formwork	436 m^2
29	1.8.3	Support beam Concreting	C25:116 m^3
30	1.8.4	Horizontal drilling	238 at 1.5 m, L=21m
31	1.8.5	Anchor install and grouting	Anchor = 4992.9 m(49744 kg), grouting= 4×238 m
32	1.9	Excavation stage 4	19650 Bm^3 (bottom level= -12.3)
33	1.10	Anchorage stage 4	
34	1.10.1	Support beam Reinforcement	7725 kg
35	1.10.2	Support beam Formwork	436 m^2
36	1.10.3	Support beam Concreting	C25:116 m^3
37	1.10.4	Horizontal drilling	238 at 1.5 m, L=19 m
38	1.10.5	Anchor install and grouting	Anchor = 4516 m, (44993 kg), grouting= 4×238 m
39	1.11	Excavation stage 5	19650 Bm^3 (bottom level= -15.3)
40	1.12	Anchorage stage 5	
41	1.12.1	Support beam Reinforcement	7725 kg
42	1.12.2	Support beam Formwork	436 m^2
43	1.12.3	Support beam Concreting	C25:116 m^3
44	1.12.4	Horizontal drilling	238 at 1.5 m, L=17 m
45	1.12.5	Anchor install and grouting	Anchor = 4039.2 m, (40243 kg), grouting= 4×238 m
46	1.13	Excavation stage 6	19650 Bm^3 (bottom level= -18.3)
47	1.14	Anchorage stage 6	
48	1.14.1	Support beam Reinforcement	7725 kg
49	1.14.2	Support beam Formwork	436 m^2

Table 9.24: Work breakdown structure for Spring Mall deep excavation (continued)

ID	WBS	Task	Definition, quantity
50	1.14.3	Support beam Concreting	116 m^3
51	1.14.4	Horizontal drilling	238 at 1.5 m, L=16 m
52	1.14.5	Anchor install and grouting	Anchor = 3806.4 m,(37923 kg), grouting= 4×238 m
53	1.15	Excavation stage 7	
54	1.15.1	excavation	11135 Bm^3 (bottom level= -20.0)
55	1.15.2	Grading	6550^{2}

9.7.4 Scheduling

Initial activity plan schedule is shown in table 9.24. Start date was 03/01/2015 and estimated finish date was 12/01/2015.

Table 9.25: Initial activity plan schedule

Task Name	Duration	Start	Finish	Predecessors
Pre-excavation	6 days	Sun 3/1/15	Fri 3/6/15	
Contiguous pile wall	85 days	Mon 3/2/15	Fri 6/26/15	2FS-5 days
Excavation stage 1-1	7 days	Fri 4/6/15	Mon 4/14/15	3FS-60days
Excavation stage 1-2	7 days	Tue 5/1/15	Wed 5/11/15	4FS+12 days
Excavation stage 1-3	7 days	Thu 6/23/15	Fri 7/1/15	5FS+20 days, 3FS-4 days
Anchorage stage 1	23 days	Thu 6/25/15	Mon 7/27/15	5FS-3 days,4, 6FS-22 days
Excavation stage 2	15 days	Thu 7/14/15	Wed 8/3/15	7FS-10 days, 6FS-2 days
Anchorage stage 2	21 days	Tue 7/28/15	Tue 8/25/15	7FS-3 days, 8FS-12 days
Excavation stage 3	15 days	Thu 8/13/15	Wed 9/2/15	9FS-10 days,8
Anchorage stage 3	20 days	Wed 8/26/15	Tue 9/22/15	9FS-6 days, 10FS-6 days
Excavation stage 4	15 days	Wed 9/9/15	Tue 9/29/15	11FS-10 days
Anchorage stage 4	18 days	Wed 9/23/15	Fri 10/16/15	12FS-8 days,11
Excavation stage 5	14 days	Wed 9/30/15	Mon 10/19/15	13FS-13 days
Anchorage stage 5	15 days	Fri 10/16/15	Thu 11/5/15	14FS-9 days, 13FS-1 day
Excavation stage 6	14 days	Tue 10/20/15	Fri 11/6/15	15FS-13 days
Anchorage stage 6	12 days	Fri 11/6/15	Mon 11/23/15	16FS-7 days,15
Excavation stage 7	12 days	Thu 11/12/15	Fri 11/27/15	17FS-8 days
Grading	3 days	Fri 11/27/15	Tue 12/1/15	18FS-1 day

task	Duration	start	finish	
Pre-excavation	6 days	Sun 3/1/15	Fri 3/6/15	
Contiguous pile wall	85 days	Mon 3/2/15	Fri 6/26/15	2FS-5 days
Excavation stage 1-1	7 days	Mon 4/6/15	Tue 4/14/15	3FS-60 days
Excavation stage 1-2	7 days	Fri 5/1/15	Mon 5/11/15	4FS+12 days
Excavation stage 1-3	7 days	Tue 6/23/15	Wed 7/1/15	5FS+20 days,3FS-4 days
Anchorage stage 1	23 days	Thu 6/25/15	Mon 7/27/15	5FS-3 days,4,6FS-22 days
Excavation stage 2	15 days	Tue 7/14/15	Mon 8/3/15	7FS-10 days,6FS-2 days
Anchorage stage 2	21 days	Tue 7/28/15	Tue 8/25/15	7FS-3 days,8FS-12 days
Excavation stage 3	15 days	Thu 8/13/15	Wed 9/2/15	9FS-10 days,8
Anchorage stage 3	20 days	Wed 8/26/15	Tue 9/22/15	9FS-6 days,10FS-6 days
Excavation stage 4	15 days	Wed 9/9/15	Tue 9/29/15	11FS-10 days
Anchorage stage 4	18 days	Wed 9/23/15	Fri 10/16/15	12FS-8 days,11
Excavation stage 5	14 days	Wed 9/30/15	Mon 10/19/1:	13FS-13 days
Anchorage stage 5	15 days	Fri 10/16/15	Thu 11/5/15	14FS-9 days,13FS-1 day
Excavation stage 6	14 days	Tue 10/20/15	Fri 11/6/15	15FS-13 days
Anchorage stage 6	12 days	Fri 11/6/15	Mon 11/23/1:	16FS-7 days,15
Excavation stage 7	12 days	Thu 11/12/15	Fri 11/27/15	17FS-8 days
Grading	3 days	Fri 11/27/15	Tue 12/1/15	18FS-1 day

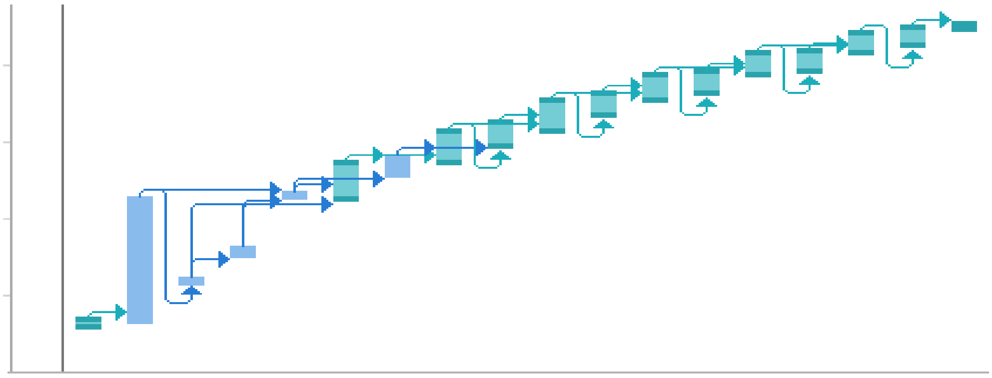

Figure 9.8: Initial linked bar chart

9.7.5 Cost estimation

Because anchorage works need to especial machines, and professional skills, this section of work was given to contractor so that the unit price for horizontal drilling, Support beam (include buying, transporting, discharge, slice, bend, close, and install of reinforcement, formwork, and labor wage of concreting), anchorage (include purchasing, transporting, and discharge of high tension steel, plates, attachments, slice, and installing), and grouting (minimum 4m length of hole) is \$23 per meter of anchorage ($4\phi20$) according to details, payment conditions, safety clauses, schedule and contract scope.

Because wells drilling for in situ piles needs to rotary bucket drilling rig, and professional skills, this section of work was given to contractor so that the unit price for each one meter vertical drilling (0.85 diameter, with loading, hauling, and discharge) is \$26 according to details , payment conditions, safety clauses, schedule and contract scope.

Since excavation needs heavy machines and professional skills, and due to volume of work which is not enough for purchasing machines in schedule, this section of work was given to contractor so that the unit price for each one bank cubic meter (include cutting, shaving, grading, loading, hauling, and discharge) is \$3.5 according to details, payment conditions, safety clauses, schedule and contract scope.

In order to precise quality control of work and saving the labor cost of inaction times, reinforcement was given to contractor with unit price of \$100 per metric ton of reinforcement (include slice, bend, close, and install). Also concrete is procured as ready production and guaranteed from concrete factory which prices were TL140 for C35 and TL120 for C25 per square meter of in-situ concrete.

Pre-excavation:

Direct cost of excavation, hauling, and dumping = 7550×\$3.5 = \$26425

Contiguous pile wall:

Direct cost of wells drilling, hauling, and dumping = 358× (21.5-1.15) ×$25= $182133

Direct cost of steel cage and installing in wells and top beam (wage) = 564.4×95 = $53618

Direct cost of steel cage buying, transport, and discharge = 400× 564.4 = $225760

Direct cost of concreting for piles and top conjunct beam = 4455.4×140/2.92= $213615

Direct cost of contiguous piles implementing = $676468

Excavation stage 1:

Direct cost of Excavation, hauling, and dumping = 15065×$3.5= $52728

Anchorage stage 1:

Direct cost of Support beam Concrete buy (C25) = 116×[(TL120)/(2.92×$/TL) = $4767

Direct cost of Horizontal drilling (238×23m), Support beam Reinforcement(7.73 ton), Support beam Formwork (436 m^2), Support beam concreting labor wage, Anchor install (54.473 ton) and grouting (min 4m) = $23×238×23m = $125902

Excavation stage 2:

Direct cost of excavation, hauling, and dumping = 19650×$3.5= $68775

Anchorage stage 2:

Direct cost of Support beam Concrete buy (C25) = 116×[(TL120/(2.92×$/TL)] =$4767

Direct cost of Horizontal drilling (238×22m), Support beam Reinforcement(7.73 ton), Support beam Formwork(436 m^2), Support beam concreting labor wage, Anchor install(52.092 ton) and grouting (min 4m) = \$23×238×22 m= \$120428

Excavation stage 3:

Direct cost of excavation, hauling, and dumping = 19650×\$3.5= \$68775

Anchorage stage 3:

Direct cost of Support beam Concrete buy (C25) = 116×[(TL120/(2.92×\$/TL)] =\$4767

Direct cost of Horizontal drilling (238×21m), Support beam Reinforcement(7.73 ton), Support beam Formwork(436 m^2), Support beam concreting labor wage, Anchor install (49.79 ton) and grouting (min 4m) = \$23×238×21 m= \$114954

Excavation stage 4:

Direct cost of excavation, hauling, and dumping = 19650×\$3.5= \$68775

Anchorage stage 4:

Direct cost of Support beam Concrete buy (C25) = 116×[(TL120/(2.92×\$/TL)] =\$4767

Direct cost of Horizontal drilling (238×19m), Support beam Reinforcement(7.73 ton), Support beam Formwork(436 m^2), Support beam concreting labor wage, Anchor install (44.99 ton) and grouting (min 4m) = \$23×238×19 m= \$104006

Excavation stage 5:

Direct cost of excavation, hauling, and dumping = 19650×\$3.5= \$68775

Anchorage stage 5:

Direct cost of Support beam Concrete buy (C25) = 116×[(TL120/(2.92×$/TL)] =$4767

Direct cost of Horizontal drilling (238×19m), Support beam Reinforcement(7.73 ton), Support beam Formwork(436 m2), Support beam concreting labor wage, Anchor install (40.24 ton) and grouting (min 4m) = $23×238×17 m= $93058

Excavation stage 6:

Direct cost of excavation, hauling, and dumping = 19650×$3.5= $68775

Anchorage stage 6:

Direct cost of Support beam Concrete buy (C25) = 116×[(TL120/ (2.92×$/TL)] =$4767

Direct cost of Horizontal drilling (238×19m), Support beam Reinforcement(7.73 ton), Support beam Formwork(436 m2), Support beam concreting labor wage, Anchor install (37.92 ton) and grouting (min 4m) = $23×238×16 m= $87584

Excavation stage 7:

Direct cost of excavation, hauling, and dumping, and grading = 11135×$3.5= $38973

Indirect cost:

Indirect costs during deep excavation are workshop office rate (let assume monthly $1200), workshop official wages including site manager (let assume monthly TL3500), surveyor (let assume monthly TL2500), accountant (let assume monthly TL2500), As-built tracer (let assume monthly TL1800), purchasing officer (let assume monthly TL1800), three guardsman (let assume monthly 3×TL1800), servant (let assume monthly TL1600), two worker (let assume monthly 2×1600), and general manager (let assume monthly TL28000), wage of welding for shoring on edge of top level beam (let assume TL3000 during deep excavation), administrative costs TL4500 per month (include electric, water, telephone, , transportation, stationery, accessories archive, kitchen).

9.7.6 Budget determination

Summary of estimated direct costs for deep excavation is indicated in table 9.26. Total direct cost is $1814359. Summary of indirect cost during deep excavation is collected in table 9.27. Total cumulative indirect cost is $180721 for nine month of schedule. Total cost is $1995080 and indirect/total cost ratio is 9%. The implemented cost budget is shown in table 9.28. The initial cost performance baseline for deep excavation is illustrated in figure 9.9 which does not contain the contingency reserve to mitigate cost risk.

Note: The cost performance baseline required adjustment so that the expenses can be smoothed and replaced from third month to second month which can reduce the share of indirect cost on total cost and reduce one month of finishing time. The adjusted cost performance baseline for deep excavation is shown in table 9.29. In that case saved cost is $20080 and saved time is one month. Adjusted cost performance baseline is illustrated in figure 9.10

Table 9.26: Summary of estimated direct costs for Spring Mall deep excavation (20 m excavation depth, 131000 cubic meters bank excavation)

Operations	**Direct Cost($)**
Excavation	462001
Pile wall (contiguous)	675126
Anchorage+ Concrete for anchors support beam	677232
Total direct cost:	$1814359

Table 9.27: Summary of estimated indirect costs for Spring Mall deep excavation

Indirect cost	**$ per month**	**$ in 9 month**	**$ Cumulative**
Workshop office rate	1200	10800	10800
Workshop official labor wages	17225	155025	165825
Welding wage for shoring		1027	166852
Administrative costs	1541	13869	180721

Table 9.28: Implemented cost budget

Operations	**1st month**	**2nd month**	**3rd month**	**4th month**
Pre-excavation	26425			
Contiguous piles wall	230795.	238753.4	206919.6	
Excavation 1-1		17576		
Excavation 1-2			17576	
Excavation 1-3				17576
Anchorage 1				28406.3
Excavation 2				
Anchorage 2				
Excavation 3				
Anchorage 3				
Excavation 4				
Anchorage 4				
Excavation 5				
Anchorage 5				
Excavation 6				
Anchorage 6				
Excavation 7				
Indirect cost	20080.1	20080.1	20080.1	20080.1
Total cost	277300.1	276409.5	244575.7	66062.4

Table 9.28: Implemented cost budget (continued)

Operations	**5th month**	**6th month**	**7th month**	**8th month**	**9th month**
Pre-excavation					
Contiguous pile					
Excavation 1-1					
Excavation 1-2					
Excavation 1-3					
Anchorage 1	102262.7				
Excavation 2	68775				
Anchorage 2	17885	107310			
Excavation 3		59605	9170		
Anchorage 3		23944.2	95776.8		
Excavation 4			68775		
Anchorage 4			48343.6	60429.4	
Excavation 5			4912.5	63862.5	
Anchorage 5				97825	
Excavation 6				54037.5	14737.5
Anchorage 6					92351
Excavation 7					38973
Indirect cost	20080.1	20080.1	20080.1	20080.1	20080.1
Total cost	209002.8	210939.3	247058	296234.6	166141.6

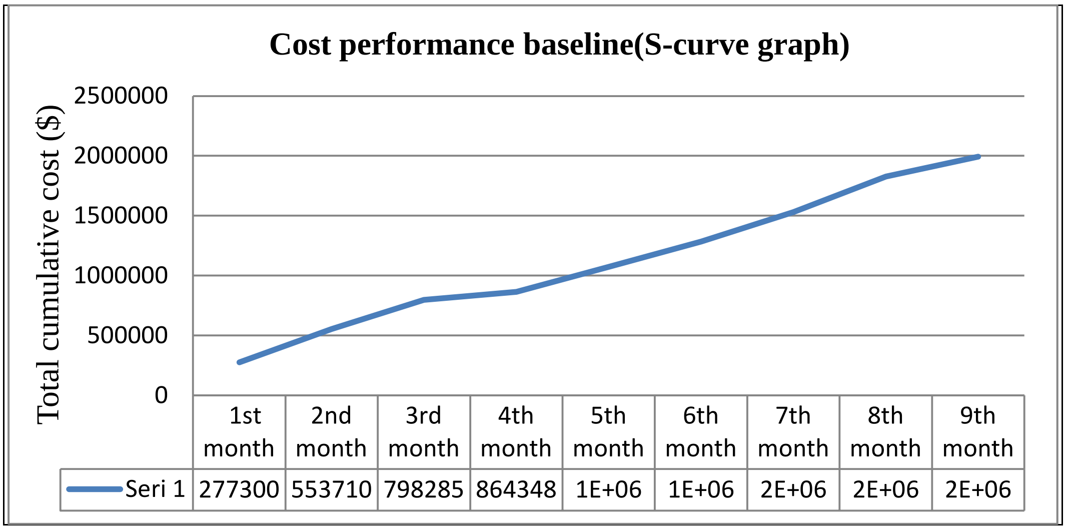

Figure 9.9: Initial cost performance baseline of Spring Mall project deep excavation

Table 9.29: Adjusted cost budget

Operations	**1st month**	**2nd month**	**3rd month**	**4th month**
Pre-excavation	26425			
Contiguous piles wall	230795.	238753.4	206919.6	
Excavation 1-1	17576			
Excavation 1-2		17576		
Excavation 1-3				17576
Anchorage 1			28406.3	102262.7
Excavation 2				68775
Anchorage 2				17885
Excavation 3				
Anchorage 3				
Excavation 4				
Anchorage 4				
Excavation 5				
Anchorage 5				
Excavation 6				
Anchorage 6				
Excavation 7				
Indirect cost	20080.1	20080.1	20080.1	20080.1
Total cost	294876.1	276409.5	255406	226578.8

Table 9.29: Adjusted cost budget (continued)

Operations	**5th month**	**6th month**	**7th month**	**8th month**
Pre-excavation				
Contiguous pile				
Excavation 1-1				
Excavation 1-2				
Excavation 1-3				
Anchorage 1				
Excavation 2				
Anchorage 2	107310			
Excavation 3	59605	9170		
Anchorage 3	23944.2	95776.8		
Excavation 4		68775		
Anchorage 4		48343.6	60429.4	
Excavation 5		4912.5	63862.5	
Anchorage 5			97825	
Excavation 6			54037.5	14737.5
Anchorage 6				92351
Excavation 7				38973
Indirect cost	20080.1	20080.1	20080.1	20080.1
Total cost	210939.3	247058	296234.5	166141.6

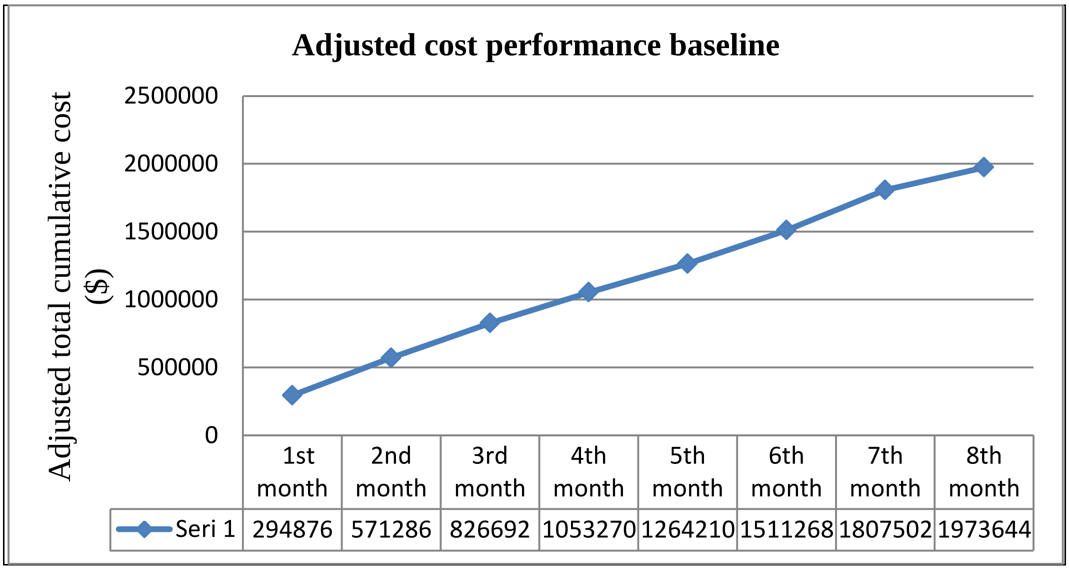

Figure 9.10: Adjusted cost performance baseline of Spring Mall deep excavation project

9.7.7 Cost and Duration risk

Real finish date was 1/20/2016 that there was delay about 50 days, which cause was collapse of auger drilling machines for contiguous piles construction and grouting machine. The minimum damage due to that delay is indirect costs which can be $20080.1\times50/30$ = \$33467 without including other expenses such as rehabilitation expenses, debt installments, contractual delay penalty, depreciation, central office overhead residual, financial costs, and the cost of lost opportunity. Also diameter of wells converted from 0.85 to 1.05 meter when measured for concreting so that instead of 4165 m3 , 5407 m3 concrete was consumed which monetary difference was \$59548. On the other hand price of steel reinforcement bars increased from \$400 to average \$450 per ton so that had \$28220 additional cost. Because of contracts, there are not machines repair or maintenance costs or other material procurement additional cost for main contractor. Till here, based on assumptions total payment and unit cost due deep excavation are:

\$1993724+\$33467+\$59548+\$28220=\$2114959

Unit cost = \$2114959/ (6550×20) = \$16.15 per Bm3

Other possible damages may be adjacent buildings and road pavement destruction as external damages. Important adjacent buildings are in five and four meters distance from excavation edge which the first one has five stories and the second has four stories respectively. Also there are two one-story buildings at three meters distance. On the other hand, the nearby road has 202 meters length which include side walk in about one meter width and after that pavement for cars as well as nearby landscapes in 130 meters length may have damages.

Road full depth patching costs is assumed \$77 per square meter of asphalt road [113] and for roadbed and other utilities such as electric or telecommunication cable or water supply \$5 per square meter of road. Damage of adjacent building depends on distance, probable ground settlement, and probable damage occurrence. Foundation settlement based on figure 7.6 for each nearby building is estimated. We assume \$550 per square meter for building area construction cost and \$130 per square meter area for nearby ordinary landscape construction cost. Based on $\Delta S/L$ ratio, and Burland damage classification (see chapter 7), possible damage of each property is guesstimated based on intuitive feelings and personal judgment in term of percent of each property's value (without land price) as shown in table 9.30.

Nearby damages payment are probable external cost which depends on nearby owner presence, act and behavior, damage occurrence, strong reason for court to deep excavation participate on damage and its contribution percent in settlement and influence of contributed settlement on building damage. It is possible that some of estimated damages are not paid unless project owner personally without going to court accept to repair or pay damage in a formal agreement morally. On the other hand it may possible all cases of damages is not occurring with high quantity so that a few are lowest, some are mean and other is high in occurrence. Thus expected payment of damage may be estimated if probability of occurrence of each estimated damage payment is known. However the optimistic and pessimistic estimated external damages are \$52598 and \$196730 respectively, based on above mentioned, expected external damage may be estimated \$136351 as described in table 9.31 which data is guesstimated based on intuitive feelings and personal judgment which may find based on lessons learnt and expertise idea. Thus the optimistic and pessimistic estimated external damage are \$0.14 and \$0.55 per Bm^3 respectively and expected external damage is \$0.38 per Bm^3 of excavation volume.

A few pictures from project are shown in figures 9.11, 9.12, 9.13, and 9.14. Steel cages for piles of wall are illustrated in figure 9.11. A view of adjacent buildings is shown in figure 9.12 which represents the 5-story building, 4-story building, 2-story building. Figure 9.13 shows third stage of anchorage implementation with two set of horizontal drill. A view of piles, anchorages, moisture isolation on pile-wall, shoring, and partial construction of main structure on 5/12/16 is illustrated in figure 9.14 which the four levels of anchorage is seen and bottom areas is covered by moisture isolation.

Table 9.30: Possible nearby damage for Spring Mall

Property	**Area** (m^2)	**Minimum distance** (m)	**Estimated settlement** (mm)	**ΔS/ L** (max)	**Damage** (%)	**Possible damage range** (\$)
5-story building	15×20	6	6 -14	0.0004	0.5 - 2	4125 - 16500
4-story building	12×20	5	6 -15	0.00045	0.5 - 2	2640 - 10560
2-story building	15×12	4	8 - 17	0.0075	0.9 - 4	1782 – 7920
2-story building	12×8	4	11 - 17	0.00213	0.8 - 3	845 - 3168
Road pavement	202×20	1	6 - 200	0.0003	1 - 5	3313 - 16564
nearby landscape	130×20	1	6 - 200	0.0003	1.5 - 5	5070 - 16900

Table 9.31: Expected external damage for Spring Mall

Property	Estimated lower damage $	Probability of lower damage occurrence	Estimated higher damage $	Probability of higher damage occurrence	Expected external damage $
5-story building	4125	0.6	16500	0.4	9075
4-story building	2640	0.35	10560	0.65	7788
2-story building	1782	0.45	7920	0.55	5158
2-story building	845	0.5	3168	0.5	2007
Road pavement	3313	0.4	16564	0.6	11264
nearby landscape	5070	0.2	16900	0.8	14534
Expected external damage :					49826

Figure 9.11: Steel cages for piles of wall

Figure 9.12: Adjacent buildings

Figure 9.13: Anchorage in stage 3

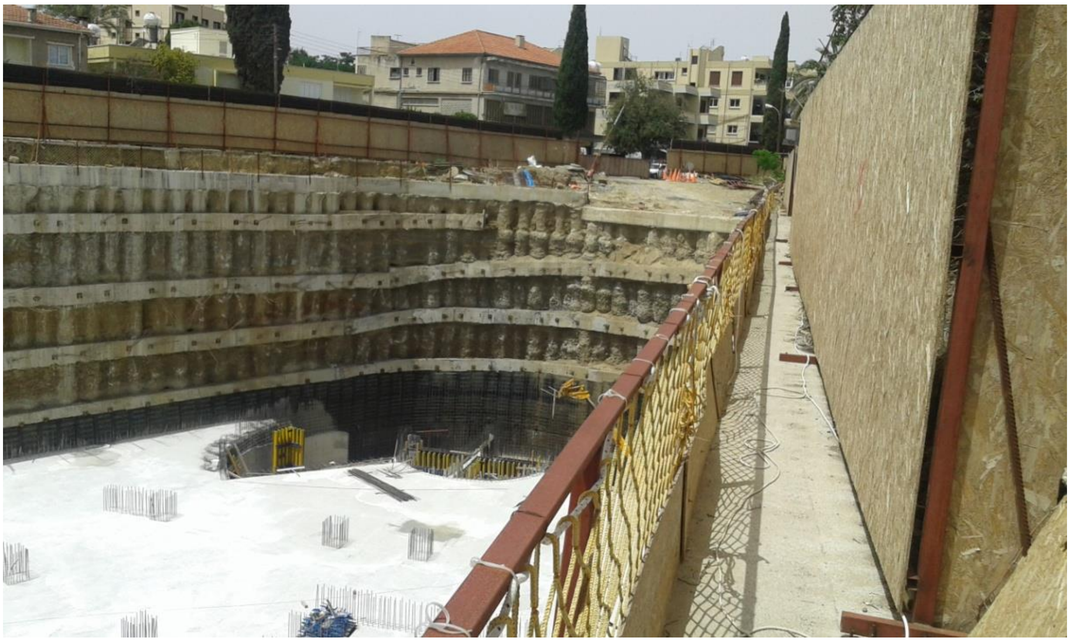
Figure 9.14: A view of piles, anchorages, moisture isolation on pile-wall, shoring, and partial construction of main structure on 5/12/16

9.8 Considerations on situation of some case histories and some others case studies in deep excavation

There are pictures or some visual collapse observations during deep excavation processes jointly with secondary multi-level supporting system. Variables such as maximum depth of excavation, Subsurface (type and levels of soils), Water table, Site area, Retaining wall (depth, thickness, material), excavation stages, Struts (levels, sizes, spans, spacing), Type of failure observations, Casualties, and causes can be important in surveying the well-documented case histories which can be gathered and collected as lesson-learnt.

Two cases of failure had reinforced concrete diaphragm wall (length = 24 m, thickness = 0.7 m for 13.45 meters excavation depth, and length = 15.4 m, thickness = 0.5 m for 9.3 meters excavation depth respectively) plus multi levels H-section steel struts [1]. There were failure observations such as large deflection of wall, bending deformation of struts, and upward movement which were in clay (CL) with high level water table. There weren't any casualties. The potential risk of deep excavation in clay with high level water table is very high and this type of works need to exactly supervising and inspection. Also design checking before construction is crucial. This consequence may be due to operational and/or construction risk.

The third case of failure had reinforced concrete diaphragm wall plus four levels pipe-section steel struts (strut layer) which had vertical steel struts (that were support for horizontal struts with about 12 and 9 meters span at 2.9 meters spacing) on bored piles (under column struts) in OL-CL soil with high level water table. There were failure observations in final stage of excavation (fifth) which strut layer was not installed at the right time which collapsed and lead to overall failure. The reinforced concrete diaphragm wall (0.70 meter thickness, 24 meters length) had large deformation so that was broken into two parts in some sections. Connections between the struts and wall failed. Heave at the excavation bottom level was expanded even 2.5 meters in certainly points. Adjacent road had subsidence about 7 meters. There had been 21 casualties [1]. The potential risk of deep excavation in clay, especially OL-CL with high level water table is very high and this type of works need to exactly supervising and inspection. Also design checking before construction is crucial. This consequence may be due to operational and/or construction risk.

The fourth case of failure had reinforced concrete diaphragm wall (44.3 meters length, 0.8 m thickness, for 33.3 meters maximum depth of excavation) plus nine levels H-section steel struts (struts layer, 10 meters span, and two spans) plus two layer jet-grout piles (1.5 meters thickness between -27.5 to -29 levels and 2.5 meters thickness between -29.4 to -32 levels. The support system collapsed at -30.6 level of excavation (tenth stage) which the wall moved inward and struts moved and distorted. Nearby road pavement subsided. There had been four casualties. Poor design of the wall-wale connection was announced as causes [1]. This consequence may be due to operational and/or construction risk.

The fifth case is about effect of deep excavation in stiff clay above the existence MRT tunnel on tunnel displacements closely under project [36]. Variables such as project building characteristics (15 story with 2 levels of basement), Excavation area, range of topographic surface levels, Excavation maximum depth (15 meters), Excavation supporting system characteristics (contiguous pile wall, diameter =1.10 m), Subsurface characteristics (SPT, Undrained shear strength, and modulus E_u), Ground water table, Minimum distance to existing tunnel, Existing tunnel characteristics (6 meters nominal external diameter, 15 to 27 meters invert depth) , Excavation duration (123 days), Tunnel lining maximum observed displacements due to effect of deep excavation on a MRT tunnel are collected which summary of case and its measured effect as maximum displacements is indicated in table 9.31. Maximum horizontal displacement is in crown of tunnel which is 6 mm and maximum vertical displacement is in spring of tunnel which is 3.8 mm. The displacements had not any important damage on function of tunnel but it is important for preventing any damages or problems before starting construction works the underground situation is studied.

Table 9.32: The effect of deep excavation on tunnel displacements [36]

Project name and location	**Project structure situation**	**Excavation area**	**Range of surface levels**	**Excavation maximum depth**	**Excavation supporting system**
Tan Tock Seng Hospital in Singapore	15 story building with 2 levels of basement	Width=140 length=200 meters	105-116 meters	15 meters	Contiguous bored pile wall with 1.10 meters diameter

Table 9.32: The effect of deep excavation on tunnel displacements [36] (continued)

Subsurface SPT, Undrained shear strength, and E_u	**Ground water table**	**Minimum distance to existing tunnel**	**Existing tunnel character**	**Excavation duration**	**Tunnel lining maximum displacement**
level : 0.0-20 m N=15 level: 20-29 m N=36 level: 29-33 m N=60 level >33m N>100 c_u = N E_u/c_u = 400	Less than 10 meters	Close	Nominal external diameter: 6 meters Invert depth: 15-27 meters	After 32 days: 5 to 9 meters After 123 days: completed	Horizontal: Invert: 3.6mm Crown: 6mm Spring: 5.8mm Vertical: Invert: 3.3mm Crown: 3.6mm Spring: 3.8mm

The sixth case is surveying about effect of underpinning of adjacent building before deep excavation, effect of retained wall construction, and effect of excavation on adjacent building by Aleksandra Chepurnova (2014) [115]. One of the solutions to deal the settlement and its effect on adjacent buildings damages is using underpinning which use micropiles and/or jet-grouting. Chepurnova's surveying was on two historical adjacent building (high value building). Summary of the surveying and results is shown in table 9.32. As it is seen the underground soil is complicate because the combination of fill, sand, loam, and sandy loam with different parameters and without thickness definition is there. Also there are two aquifers at depth of 30 meters; however it is deeper than the excavation depth (11 meters) and there isn't any impermeable soil above that for introducing upheaval. The adjacent buildings have one basement and this subject reduces the effect of excavation depth from 11 meters to about 8 meters for sliding, and overturning failure. In case number 1, the settlement introduced by underpinning is about 56.5% of total induced settlement while for diaphragm wall construction it is 24.7% and for excavation it is 18.2%. Also in case number 2, the settlement introduced by underpinning is about 54.4% of total induced settlement while for diaphragm wall construction it is 34.4% and for excavation it is 16.8%. The differences between the two cases are: 4-story plus one basement vs. 9-story plus one basement, different underpinning system. The effect of different stories may affect the soil bearing capacity. Also there are differences in stiffness of each underpinning methods and there is need to more study about their stiffness. However, It seems we may assume the range of 54.4% to 56.5% of total induced settlement for underpinning, 24.7% to 34.4% of total induced settlement for diaphragm wall construction, 16.8% to 18.2% of total induced settlement for excavation in similar cases (according the table parameters such as distance, soil type and so on) for initially risk assessment of adjacent building which selected to strengthening by underpinning. Also it seems the expense of underpinning is not low relatively but it is a dealing way to prevent more damages. Even there may be periodic or duplicate maintenances of adjacent buildings.

Table 9.33 Effect of deep excavation on adjacent historical buildings [115]

N	Case	Project building state	Subsurface (top to bottom respectively) γ (kN/m³), ϕ (°) , c (kPa), E (MPa)	Excavat depth
1	The new Block B of the Historical Complex in the center of Moscow	6-storey building with three basement	Fill: 18 , 8 , 18 , 13 Sand: 16.6-17.6 , 30-32, 1 ,23-29 Loam: 20.1-20.4, 12-18, 34-55 , 25-32 Sandy loam: 19.8 , 25 , 17 , 18 Sand: 16.7-18.4, 30-33, 1 ,26 - 31 Sand: 18.5 , 34 , 1 , 28 Loam: 19.3 , 12 , 31 , 14 The layers level and thick varies. 2 aquifers are observed at the depth of 30 m	11 meters Three stages of excavat.
2	The new Business and Cultural Center of the Historical Complex in the center of Moscow		Fill: 18 , 8 , 18 , 13 Sand: 16.6-17.6 , 30-32, 1 ,23-29 Loam: 20.1-20.4, 12-18, 34-55 , 25-32 Sandy loam: 19.8 , 25 , 17 , 18 Sand: 16.7-18.4, 30-33, 1 ,26 - 31 Sand: 18.5 , 34 , 1 , 28 Loam: 19.3 , 12 , 31 , 14 The layers level and thick varies. 2 aquifers are observed at the depth of 30 m	11 meters Three stages of excavat.

Table 9.33: Effect of deep excavation on adjacent historical building [115] (continued)

N	Supporting system	distance from the external structure	Nearby building	Activity for preventing excessive settlements of the adjacent structures
1	diaphragm wall thickness: 0.8 m length: 20-22 m + struts	2 meters	Block A of the Historical Complex in the center of Moscow which had 4 story with a basement	Underpinning the pad and the strip foundation by reinforcing with four cross mini-jet elements (ϕ 300 mm) then the jet-grouted columns (ϕ 800 mm), four on each pad foundation
2	diaphragm wall thickness: 0.6 m depth: m + struts	1-1.2 meter	9-storey masonry apartment houses and historical Building and palaces	Underpinning by micropiles ϕ 159-180 mm, and jet-grouting columns ϕ 600 mm (jet-1) as a joint sealing between them

Table 9.33: Effect of deep excavation on adjacent historical building [115] (continued)

N	Consumed Materials for adjacent structures	Underpinning-induced settlement (maximum)	Retaining wall-induced settlement (maximum)	Excavation-induced settlement (maximum)	Total induced settlement (maximum)
1	A length of about 37 km was grouted (jet-1); about11,500 ton cement was used, and a length of 18.3 km steel (ϕ57 mm) bars was installed, within 3 months.	4.8 mm	2.1 mm	1.6 mm	8.5 mm
2		6.8 mm	4.3 mm	2.1 mm	12.5 mm

One of the related questions is estimating effect of deep excavation activities or procedures on seismic exposure of adjacent buildings. Castaldo et al (2014) estimated the belongings of deep excavation stages as pre-excavation and past excavation on seismic vulnerability of existing eight stories reinforced concrete framed structures in sandy soils with high level water table [14]. In this type of studies after estimating settlement of foundation of adjacent building due to each excavation stage and seismic loading (by PLAXIS software or other accepted techniques), the structure of building is analyzed for that settlements jointly the seismic loading and inter-story drifts at each level related to each stages effects such as pre-excavation and/or past excavation is calculated and compared with building code limits as a damage index.

9.9 Considerations on risk assessment of production rate and duration

As an example let assume retaining wall trenching that is done by clamshell. Both clamshell and auger drill machines are not used for loading of excavated soil. Loading of excavated soil for retaining wall after separating of bentonite is done by loader or hydraulic excavator. Excavation without loading might has risky factors which influence on work productivity and scheduling such as weather condition, maneuvering, mechanical breakdowns, and operator efficiency. Furthermore the capability of selected machine which is reflected in its characteristics and bucket size has impact on work productivity and scheduling as another risky factor. The pointed facts produce additional hidden costs and time which have to attend in planning or even during construction for cost and time saving. In a normal situation, weather condition, maneuvering, mechanical breakdowns, and operator efficiency can influence 10%, 8%, 5%, and 7% on time of working [17] plus accepted normal 10 minutes per 60 minutes for rest and machine maintenance on time (50/60). Weather condition may divide into favorable (without decreasing impact) and unfavorable. Maneuvering may divides into favorable (without decreasing influence) and unfavorable. Mechanical breakdowns may depend on machine state that may be new machine (without decreasing impact) and used machine which is repaired. Operator efficiency may divide into favorable (without decreasing influence) and unfavorable which is human activity and depends on skill, training, job dissatisfaction, culture, and other human characteristics. Thus, the job efficiency can be estimated for different 16 cases by conditional probability which is indicated in table 9.33. The range of job efficiency is between 0.61 and 0.833 but what is expected job efficiency, and is it mean of the 16 cases job efficiency. It seems it is depended on optimistic, mean, and pessimistic situation. The job efficiency for optimistic, mean, and pessimistic case is 0.61, 0.715, and 0.833. Thus weather condition, maneuvering, mechanical breakdowns, and operator efficiency have potential risk and identified risk for trenching of diaphragm wall.

If a crawler clamshell with 227 kW rated power is rented $121 per hour [116] and 1.77 Lm3 hydraulic grab (780 mm width and 2500 mm length) is rented $6.75 per hour [116] which they are the responsibility of the renter, is it enough for rent contract? Of course the answer is no because there is need to other conditions had to be satisfied which are production rate, scheduling, transport on site, slurry tank and centrifugal pump coordination responsibility, and other duties. The ideal contract defines the responsibilities, duties, and obligations of each of the client, the contractor, and the consulting engineer [65]. Contractor of underground works is often faced with more uncertain site condition resulting in significant risks which includes potential damage to the properties of third parties, risk to the contractor's labor force and equipment, and risk to partially completed works of the employer [65]. There is need to clarify state of other related subjects to avoid future lawsuits and dispute if it is possible by negotiate share percent of risks and reach to appropriate agreement or by bidding.

Production rate and time of construction have impact on cost and scheduling as identified risks. Let assume based on producer of crawler clamshell catalogues winch line hoisting speed is 115 m/min, jaws opening and closing time is 17 second, maximum effective hoisting force 500kN and maximum travel speed is 1.5 km/hr [16]. Also let assume five case histories [34, 35, 36] geometry of deep excavation with different length of diaphragm wall and different excavation volume in clay for comparing which is indicated in table 9.34. Nominal cycle time can be calculated by dividing mean of wall length on winch line hoisting speed (115 m/min) and multiplication to mean of wall length which multiplication is multiplied to 60 minutes for obtaining excavation nominal cycle time. Also because of machine relocation from each 8 to 15 cycles, let assume 7 seconds added to nominal cycle time. Nominal cycles per hour are obtained by dividing of 3600 on nominal cycle time. Grab capacity as a nominal volume per cycle and its fill factor, job efficiency, and nominal cycles per hour multiplication gives production in term of loose cubic meter per hour (Lm^3/hr). There are at least 16 cases for job efficiency as mentioned in range of 0.61 to 0.833, range of grab fill factor in clay from 0.50 to 0.95 [20], and clay swell factor range from 1.25 to 1.4 [20], which are identified risk for production, minimum duration of work, and rent cost. For example optimistic estimating of grabbing production, minimum duration, and rent cost with 0.833 for job efficiency, 0.95 for grab fill factor, and 1.25 for clay swell factor is shown in tables 9.35 and 9.36. The pessimistic estimating of grabbing production, minimum duration, and rent cost with 0.61 for job efficiency, 0.50 for grab fill factor, and 1.4 for clay swell factor is shown in tables 9.37 and 9.38. The tables 9.35 and 9.36 are starting from estimating nominal cycle time, nominal cycles per hour, then apply grab nominal volume per cycle (Loose m^3), grab fill factor, and estimated job efficiency to reach Loose m^3 production while the real production relates to bank m^3 of trenching. Therefore the tables 9.37 and 9.38 are required which apply swell factor, estimates bank m^3 production, and with applying trench total volume (bank m^3), in conclusion the minimum duration (hr) and rent cost are estimated.

The comparison of the tables 9.36 and 9.38 shows the significant impact of the three risky variables on production so that optimistic estimated production is near to three times higher than pessimistic , optimistic estimated minimum duration is almost one-third of pessimistic, and optimistic estimated rent cost is approximately one-third by pessimistic. This clarifies the significant cost difference for a unit work

While risk cost is defined equal to multiplication of probability of event occurrence, period of event on unit of period, and cost of delay [52] which defines the value of the risk item as cost of maintaining the site with no work in progress but in situations such as this example the difference between optimistic estimated cost and pessimistic estimated cost is a risk cost potential interval due to situation. The mentioned deviation and impacts can be seen in unit cost of trenching share in deep excavation cost due to direct rent per hour of clamshell and hydraulic grab which is shown in table 9.39.

Table 9.34: Influence of conditions on job efficiency of clamshell or auger drill in retaining wall excavation

Case	**Weather condition**	**Maneuvering**	**Mechanical breakdowns**	**Operator efficiency**	**Job efficiency**
1	Favorable	Favorable	New machine	Favorable	0.833
2	Favorable	Favorable	New machine	Unfavorable	0.775
3	Favorable	Favorable	Used machine	Favorable	0.792
4	Favorable	Favorable	Used machine	Unfavorable	0.736
5	Favorable	Unfavorable	New machine	Favorable	0.767
6	Favorable	Unfavorable	New machine	Unfavorable	0.713
7	Favorable	Unfavorable	Used machine	Favorable	0.728
8	Favorable	Unfavorable	Used machine	Unfavorable	0.677
9	Unfavorable	Favorable	New machine	Favorable	0.75
10	Unfavorable	Favorable	New machine	Unfavorable	0.698
11	Unfavorable	Favorable	Used machine	Favorable	0.713
12	Unfavorable	Favorable	Used machine	Unfavorable	0.663
13	Unfavorable	Unfavorable	New machine	Favorable	0.69
14	Unfavorable	Unfavorable	New machine	Unfavorable	0.642
15	Unfavorable	Unfavorable	Used machine	Favorable	0.656
16	Unfavorable	Unfavorable	Used machine	Unfavorable	0.61

Table 9.35: Five case histories of deep excavation with different length diaphragm wall and different excavation volume in clay for comparing

Case	Length	Width	Maximum Depth of excavation	Diaphragm wall	Volume of excavation
1	34 m	8 m	14.0 m	Length= 32 m Thickness= 0.8	3800 Bm^3
2	16 m	20.5 m	13.1 m	Length=31 m Thickness=0.8 m	4200 Bm^3
3	43 m	19.3 m	15.2 m	Length=35m Thickness=0.8 m	12000 Bm^3
4	85.5 m	23.6 m	28.0 m	Length= 50 m Thickness=2.0 m	54000 Bm^3
5	443.9 m	44.5 m	32.0 m	Length=40m Thickness= 0.8	430000 Bm^3

Table 9.36: Optimistic estimating of hydraulic grabbing production (Loose m^3)

Case	**Nominal cycle time** (sec)	**Nominal cycles per hour**	**Nominal volume per cycle** (Lm^3)	**Grab fill factor**	**Job efficiency**	**Production** (Lm^3/hr)
1	33.7+7	88.452	1.77	0.95	0.833	123.89
2	33.2+7	89.55	1.77	0.95	0.833	125.43
3	35.3+7	85.1	1.77	0.95	0.833	119.2
4	47.3+7	66.298	1.77	0.95	0.833	92.86
5	37.9+7	80.178	1.77	0.95	0.833	112.3

Table 9.37: Optimistic estimating of hydraulic grabbing production, minimum duration and rent cost

Case	**Swell factor**	**Production** (Bm^3/hr)	**Trench total volume** (Bm^3)	**Minimum duration** (hr)	**Rent cost** ($)
1	1.25	99.11	2150	21.69	2882
2	1.25	100.34	1810	18.04	2397
3	1.25	95.36	3488	36.58	4860
4	1.25	74.29	21820	293.72	39024
5	1.25	89.84	31257	347.92	46225

Table 9.38: Pessimistic estimating of hydraulic grabbing production (Loose m^3)

Case	**Nominal cycle time** (sec)	**Nominal cycle per hour**	**Nominal volume per cycle** (Lm^3)	**Grab fill factor**	**Job efficiency**	**Production** (Lm^3/hr)
1	33.7+7	88.452	1.77	0. 5	0.61	47.75
2	33.2+7	89.55	1.77	0. 5	0.61	48.34
3	35.3+7	85.1	1.77	0. 5	0.61	45.94
4	47.3+7	66.298	1.77	0. 5	0.61	35.79
5	37.9+7	80.178	1.77	0. 5	0.61	43.28

Table 9.39: Pessimistic estimating of hydraulic grabbing production, minimum duration and rent cost

Case	**Swell factor**	**Production** (Bm^3/hr)	**Trench total volume** (Bm^3)	**Minimum duration** (hr)	**Rent cost** ($)
1	1.4	34.11	2150	63.04	8054
2	1.4	34.53	1810	52.42	6697
3	1.4	32.81	3488	106.3	13580
4	1.4	25.56	21820	853.54	109040
5	1.4	30.91	31257	1011.09	129167

Table 9.40: The range of three risky factors, production, minimum duration, and unit cost of trenching share in deep excavation cost due to direct rent per hour of clamshell and hydraulic grab

Case	**Grab fill factor range**	**Job efficiency range**	**Swell factor range**	**Production range** (Bm^3/hr)	**Minimum duration range** (hr)	**Unit cost of trenching share in deep excavation cost due to direct rent per hour of clamshell and hydraulic grab range** ($/$Bm^3$)
1	0.5-0.95	0.61-0.833	1.4-1.25	34.1-99.1	63 – 21.7	0.76-2.12
2	0.5-0.95	0.61-0.833	1.4-1.25	34.5-100.3	52.4 - 18	0.57-1.60
3	0.5-0.95	0.61-0.833	1.4-1.25	32.8-95.4	106.3 – 36.6	0.41-1.13
4	0.5-0.95	0.61-0.833	1.4-1.25	25.6-74.3	853.5-293.7	0.72-2.02
5	0.5-0.95	0.61-0.833	1.4-1.25	30.9-89.8	1011.1-347.9	0.11-0.30

As it is seen there could be situations that grab fill factor can change between 0.5 and 0.95, job efficiency can change between 0.61 and 0.833, swell factor can change between1.4 and 1.25 which can create risk in production rate, work duration, and unit cost of trenching share in deep excavation because can affect the outlook of project objectives in a particular period of a process. These situations must deal by accurate controlling so that the consequence tends to favorable initial estimation. Lack of control may lead to not only low level of the three mentioned factors, but also may lead to very poor result and irrecoverable consequence.

Similar the grabbing and with more factors there are risky situations for excavation by hydraulic backhoe so that kind of component set, job efficiency and hauling of soil can affect the production rate, work duration, and unit cost of excavation which affect the outlook of project objectives in a particular period of a process. Loading of excavated soil is done by hydraulic backhoe commonly unless there isn't possible to construct temporary ramp into deep due to lack of location which needs to benching so that lower hydraulic backhoe excavate and dump the soil on upper level of bench and other hydraulic backhoe translate the soil to next upper level of bench to reach to ground level and load to dump truck.

The situation which loading of excavated soil is done by hydraulic backhoe can be affected by job efficiency and hauling of soil. Job efficiency can be affected by weather condition about 10% [17], maneuvering approximately 8% [17], mechanical breakdowns about 5% [17], operator efficiency nearly 7% [17], and waiting for dump trucks approximately 10% [17] so that job efficiency can tolerate between 0.833 and 0.45 depends on situation. Hauling of soil can affected by factors including: Site conditions, volume of soil to be moved, type of soil, time available, maximum payload of bucket and truck, compatibility of backhoe and truck (maximum loading height, reach and truck maximum height), management status (bonus target, overtime, etc.), and road conditions (grad, altitude, pavement type, cleaning of road, and capacity). However, the factors effects on hauling can be seen in cycle time of hauling.

Cycle time of hauling is sum of fixed time and variable time which fixed time includes: spot time (moving the unit into position to begin loading), load time, maneuver time, and dump time. Variable time represents the travel time require for a unit to haul soil to the delivery site and return. Travel time depends on the grades faced, the altitude above sea level, the filled hauler's weight in ascending, the empty hauler's weight in descending, engine power, and the conditions of the hauling road [20, 26]. For travel time, the below relationship is considering for maximum speed determining [20, 26]:

$$\text{Effective grade} = \text{grade (\%)} + (0.1\times \text{Rolling Resistance factor})$$

where rolling resistance factor value is function of tire type and road surface condition which is selected from table 9.40 [20, 26]. The coefficient of (100 - Effective grade)% is multiplied on maximum speed of hauling unit in order to reduce it in ascending which maximum speed of hauling unit can determine from catalogue or tachometer. The grade (%) of road can be found by surveying or existence road map which can divide into sections with different grades. Crawler mounted vehicles has no rolling resistance to consider. However, if crawler mounted tractor tows a tired vehicle, the rolling resistance of the towed vehicle will be considered. Grade resistance is positive when the vehicle is travelling up a grade and negative when it is travelling downhill.

Internal combustion engine power rating of hauling unit falls nearly 3% for each approximately 300 meters raise in altitude above the reference altitude (e.g. sea level) at where full rated power is delivered [20, 26] which affects the hauling unit nominal payload [25]. For travel time, the below relationship is considering for internal combustion engine power rating of hauling unit determining:

Derating factor (%) = (Altitude - 915) / 102

Percentage of rated power available =100 – Derating factor

Hauler real payload = Nominal payload × Percentage of rated power available

Required payload of hauler per cycle is determined by dividing required hourly production on the number of cycles per hour. After required payload per cycle has been calculated, the payload should be divided by the loose cubic meter soil weight to determine volume of loose cubic meter required per cycle. A reasonable knowledge of soil weight is necessary for accurate estimation of production. After determining volume of loose cubic meter required per cycle of hauling, the bucket size of excavator or loader can be estimated by:

Bucket size = (volume required / loading cycle) / (bucket fill factor)

Loading cycle time of hydraulic excavator is sum of digging and load bucket (0.08 – 0.12 minutes), swing loaded (0.04 – 0.14), dump bucket (0.02 – 0.04), and swing empty (0.05 – 0.14), which total loading cycle time is between 0.21 and 0.44 minutes [25]. Loading cycle time of track loader (load, maneuver, and dump) is between 0.25 and 0.35 minutes for a small or medium loader which load time is between 0.03 and 0.07 minutes, maneuver time about 0.2 minutes, dump time between 0.04 and 0.07 minutes [25]. When soil is hard or loader is large, basic cycle time's increases until 0.65 minutes [17]. The below formulas are used for load time, number of haulers required, and expected production without influence of road capacity and traffic in situations [17].

Load time = haul unit capacity / loader production

Load time = Number of bucket loads × excavator cycle time

Number of haulers required (N) = (Haul unit cycle time) / (load time)

Expected production = [(Actual number of units) / N] × Excavator production

Actual payload of hauler has to be integer multiplication off excavator bucket payload in order to maximum use of hauler. For example five pass of excavator bucket can fill 100% hauler actual payload which can be examined by weighing of unfilled and filled hauler on weighbridge otherwise the hauler is used by e.g. 90% efficiency instead of 100% which can influence on increasing of indirect costs, hauling cost, probability of accidents, probability of pavement damage, and air pollution. For that reason there is crucial to state a penalty in contract of hauler to observance of 100% using of payload.

Production of hauling is limited by road capacity which affects the travel time. Road capacity depends on stopping distance plus length of vehicles, and maximum speed. Stopping distance is the distance vehicle travels from the time which driver sees a hazard and press on the brake until the vehicle stops. Stopping distance is made up of perception distance, reaction distance, and braking distance. Perception distance is the distance which a vehicle travels while a driver is identifying, predicting and deciding to slow down for a hazard. The distance the vehicle travels while driver react is called a reaction distance which duration is about 2.5 seconds. Braking distance is the distance a vehicle travels from the time a driver begins pressing on the brake pedal until the vehicle comes to a stop. For trucks, the brake lag distance in the stopping distance is added. Brake lag is the time it takes for a brake signal to travel to all the wheels on the tractor-trailer (0.75 second). Brake lag distance is the distance the truck travels before the brakes on the trailer are engaged [117, 118]. Based on mentioned a formula for estimating of road capacity is proposed. The estimated stopping distance is minimum distance while heavy traffics can surge the cycle time of loading.

Capacity of road = Length of road / (Length of vehicle + Stopping distance)

Stopping distance = $V\times T_1 + [V^2/ (2\times g\times \mu)] + V\times T_2$

where $T_1 = 2.5$ second (Driver reaction time), $T_2 = 0.75$ second (Brake lag distance time), V= vehicle speed when driver begins pressing on the brake,

g = acceleration of gravity, μ = friction coefficient (table 8.41)

Table 9.41: Rolling resistance factor value (kg/ton) vs. tire type and road surface condition [20, 26]

Surface condition	**Conventional tires**	**Radial tires**
Asphalt or Concrete	20 + [6 × penetration in cm]	15 + [6 × penetration in cm]
Firm, smooth, flexing slightly under load	32 + [6 × penetration in cm]	26 + [6 × penetration in cm]
Rutted dirt	50 + [6 ×penetration in cm]	50 + [6 × penetration in cm]
Soft, rutted dirt	75 + [6 ×penetration in cm]	75 + [6 × penetration in cm]
Loose sand or gravel	100 + [6 ×penetration in cm]	100 + [6×penetration in cm]
Soft, muddy, deeply rutted (β =150 to 200)	β + [6 ×penetration in cm]	β + [6 ×penetration in cm]

Table 9.42: Friction coefficient (μ) for kind of road surfaces conditions [119, 120]

Road surface	**Dry, V≤50km/hr**	**Dry, V>50km/hr**	**Wet, V≤50km/hr**	**Wet, V>50km/hr**
Asphalt or Tar:				
New, Sharp	0.80 - 1.20	0.65 - 1.00	0.50 - 0.80	0.45 - 0.75
Traveled	0.60 - 0.80	0.55 - 0.70	0.45 - 0.70	0.40 - 0.65
Traffic Polished	0.55 - 0.75	0.45 - 0.65	0.45 - 0.65	0.40 - 0.60
Excess Tar	0.50 - 0.60	0.35 - 0.60	0.30 - 0.60	0.25 - 0.55
Portland Cement:				
New, Sharp	0.80 - 1.20	0.70 - 1.00	0.50 - 0.80	0.40 - 0.75
Traveled	0.60 - 0.80	0.60 - 0.75	0.45 - 0.70	0.45 - 0.65
Traffic Polished	0.55 - 0.75	0.50 - 0.65	0.45 - 0.65	0.45 - 0.60
Gravel:				
Packed, Oiled	0.55 - 0.85	0.50 - 0.80	0.40 - 0.80	0.40 - 0.60
Loose	0.40 - 0.70	0.40 - 0.70	0.45 - 0.75	0.45 - 0.75
Ice:				
Smooth	0.10 - 0.25	0.07 - 0.20	0.05 - 0.10	0.05 - 0.10
Snow:				
Packed	0.30 - 0.55	0.35 - 0.55	0.30 - 0.60	0.30 - 0.60
Loose	0.10 - 0.25	0.10 - 0.20	0.30 - 0.60	0.30 - 0.60

Chapter 10

CONCLUSION

Factors such as site geometry, main foundation & structure plan, type of local soil, ground water level, steadiness of underground, environmental conditions, occupancy safety, allowable construction period, investment or finance to high income or solving social problems, neighboring utilities, and existing adjacent buildings influenced on deep excavation process and methods, but societal or economical profits for land owner such as the high price of real estate, limit of space by property lines, lack of inadequate space at city centers, and compulsory transporting station point may lead to use vertical cutting in more than about 4 meters to even 40 meters and more depths.

A methodology to study risk management in deep excavation is proposed in chapter 4. Risk management in construction of deep excavation includes risk identification which is often negative, risk assessment, risk response plan and strategies for decision-making, so that how it is avoided, transferred, mitigated, or accepted and deal with.

Kind of construction methods, equipments, and instruments in deep excavation are summarized and classified by a proposed facade vision which are first facade such as diaphragm wall, kind of bored piles wall, steel sheet pile, and soldier beam and lagging, second facade one level support with two stages excavation such as struts, anchorage and slab of building main frame, and third facade multi-level multi-propped support with multi stages excavation such as struts layers in different levels, anchorage layers in different levels, slabs of building main frame in different levels, and combination of mentioned methods in chapter 2. Geometry of historical deep excavation implemented sites is classified.

Damage division into internal, external, and accident is proposed in chapter 3. A method for classifying and estimating expected internal and external damages in deep excavation is proposed briefly which its formulas for cost and time is proposed for deep excavation in chapter 3. Illustrative examples 9.6 and 9.7 have this proposal application.

A deterministic method for underground risk analyzing is proposed based on geotechnical engineering parameters and models which uses factor of safety by a proposed formula to estimate probability of geotechnical risk occurrence in chapter 4. Illustrative example 9.6 uses this proposal.

In order to risk identifying, site investigation and underground identification is overviewed. The range of underground soil data for clays, granular, and intermediate soils is collected and prepared based on existence pointed references which may be new approach for underground risk identification and analyzing. This approach is used in illustrative example 9.6. Also the influence of underground soil type as a risky factor on design of diaphragm wall is described in illustrative examples 9.2, 9.3, and 9.4 which use the proposed developed Bells method in chapter 6.

Bells method is developed for multi layers underground soils in order to simplifying the sliding analysis of deep excavation.

In order to risk identification and analysis, different underground geotechnical and structural failure modes (22 types) such as sliding, overturning, bearing capacity, basal heave, upheaval, liquefaction, heaven, piping, sand boiling, and another ground failure modes which collected altogether and failure effects such as adjacent building settlements and cracks or nearby road pavement settlements as well as dewatering is overviewed and summarized in chapters 6, 7, and 8 respectively which are used for risk register, analyzing and estimating expected internal, and external damages that proposed in chapter 3 and is used in illustrative example 9.6.

By proposed expanded FOS formula, probability for sliding occurrence in each stages of excavation for situation after diaphragm wall and before strut construction is estimated which is indicated in table 9.3.

Possible internal damage and expected internal damage due sliding per meter of site (wall) perimeter for each stage of excavation are estimated that is shown in table 9.4 which includes soil, reinforced concrete slide wall demolish, reinforced concrete wall reconstruct, early works, overhead, and equipment.

Expected internal damage \$430, \$3112, \$7110, \$10086, and \$13885 per meter of site (wall) perimeter for 3rd, 4th, 5th, 6th, and 7th stage of excavation is estimated respectively which calculation is indicated in table 9.4.

Possible external damages and expected external damage due sliding per meter of site (wall) perimeter for each stage of excavation are estimated that is shown in table 9.5 which includes road and adjacent building.

Expected external damage \$66.3, \$1516.8, \$7725.6, \$25314.8, and \$79667 per meter of site (wall) perimeter for 3rd, 4th, 5th, 6th, and 7th stage of excavation is estimated respectively which calculation is indicated in table 9.5.

Based on results, Internal damage shares (percent) in term of excavation stage for sliding is reduces from upper levels to bottom levels stages of excavation whereas external damage shares (percent) in term of excavation stage for sliding is increased which are illustrated in figures 9.4 and 9.5.

Impact of expected damages due sliding on project is estimated which are 1.99%, 1.99%, 2.73%, 21.31%, 63.76%, 96.32%, and 137.12% due to sliding and accident in 1st, 2nd, 3rd, 4th, 5th, 6th, and 7th stage of excavation respectively. This subject is indicated in table 9.6.

Optimistic and pessimistic possible damages and impact on project due sliding and accident after diaphragm wall construction due 1st, 2nd, 3rd, 4th, 5th, 6th, and 7th stage of excavation is estimated which indicated in table 9.7. Expected damage in 1st and 2nd stages of excavation is due to accident alone which monetary damage is \$256109 with 1.99% impact on project. Expected damage in 3rd stage of excavation is due to sliding and accident which optimistic and pessimistic are \$382633, and \$410749 with 2.97% and 3.19% impact on project respectively. Expected damage in 4th stage of excavation is due to sliding and accident which optimistic and pessimistic are $2670988, and $3207629 with 20.75% and 24.92% impact on project respectively. Expected damage in 5th stage of excavation is due to sliding and accident which optimistic and pessimistic are $7675726, and $9324530 with 59.64% and 72.45% impact on project respectively. Expected damage in 6th stage of excavation is due to sliding and accident which optimistic and pessimistic are $11517156, and $14019611 with 89.46% and 108.93% impact on project respectively. Expected damage in 7th stage of excavation is due to sliding and accident which optimistic and pessimistic are $16320046, and $19889810 with 126.81% and 154.54% impact on project respectively.

According to results, overturning is determinative for selection of struts and slab requested structural design and construction due to low factor of safety which additional resistant moment is calculated by applying recommended sure factor of safety on overturning moment and reducing existence resistant moment. For example in level -17.2 the overturning moment and resistant moment are 28216 and 1123 kNm/m respectively but the additional requested resistant moment is 34147 kNm/m. The requested resistant force for that moment is 13134 kN/m which need to 71 cm^2/m steel cross-section that has to have enough moment of inertia to deal with span bending due own weight.

The result of analysis shows that there is Basel heaves in excavation stages of illustrative example 9.5 which is removed in next stage of excavation with slightly increasing in the excavation volume and causes very slightly deformation of nearby lands and settlement under adjacent foundation. To deal and reduce or even eliminate the Basel heave, reducing of surcharge is effective; especially it is reduced from 100 kN/m^2 to 13 kN/m^2 the elimination of heave will be realized if it is possible in practice. Also main foundation's weight must raise FOS up to1.2 for case of surcharge existence.

The result of analysis shows that there is bottom heave due unloading in excavation stages of illustrative example 9.6 that is removed in next stage of excavation which slightly increases the excavation volume and causes deformation of nearby lands and settlement under adjacent foundation. If depth of excavation is bigger than width of excavation it is possible to reduce or even eliminate the heave by reducing surcharge (to a certain extent), otherwise it is inevitable or large loading is required. Large loading is expected after construction of building then at the time of excavation execution it is inevitable and should be accepted or discouraged the project.

Heaven due artesian pressure will be in stages 4th until 7th of excavation which is inevitable at the time of excavation and its back expected after construction of main foundation or even building and should be accepted or discouraged the project of illustrative example 9.6.

Hydraulic failure will be in stages 5th until 7th of excavation of illustrate example 9.6. To mitigate and deal with the hydraulic failure risk there is need to dewatering before excavation (even a few months in clays) and during excavation so that the water table comes under excavation level. On the other hand, dewatering consequences is settlement of soils under adjacent foundation, road, utilities, and landscapes. Moreover without dewatering, hydraulic failure is inevitable at the time of excavation and it is expected after construction of main foundation or even building and should be accepted or discouraged the project unless freezing of pore water technology is used which cost is high and it is request to survey and comparing alternatives cost.

A Geotechnical risk response plan for a top-down method is prepared and proposed in table 9.19 for illustrative example 9.6.

Some of cost risks in construction of multi-level anchored contiguous bored piles wall as supporting system with seven stages excavation in Northern Cyprus is studied which results is presented in illustrative example 9.7. Work breakdown structure, scheduling, cost estimation which divided into direct and indirect costs, budget determination, cost performance baseline, adjusted cost budget, adjusted cost performance baseline, minimum damage due to delay, additional material consuming cost, material price fluctuation impact, possible external damages ranges, discussion about situation of external damages, optimistic and pessimistic estimated external damages, expected external damage, and unit cost of the deep excavation are studied.

Collapsed machine caused delay about 50 days which caused at least $33467 additional indirect cost as damage in a deep excavation project with estimated total direct cost of $1814359 that impact is 1.85% of estimated total direct cost (illustrative example 9.7).

An adjusted cost performance can save at least $20080 in cost and one month duration in a deep excavation project with total direct cost of $1814359 that impact is 1.11% of estimated total direct cost (illustrative example 9.7).

Additional concrete consuming in piles has $59548 additional cost in a deep excavation project with estimated total direct cost of $1814359 that impact is 3.28% of estimated total direct cost (illustrative example 9.7).

Steel bar price increase has $28220 additional cost in a deep excavation project with estimated total direct cost of $1814359 that impact is 1.56% of estimated total direct cost (illustrative example 9.7).

For 131000 Bm^3 by multi level anchorage contiguous pile wall supporting system deep excavation, $2114959 cost has paid which unit cost is $16.15 per Bm^3.

The optimistic and pessimistic estimated external damages are $0.14 and $0.55 per Bm^3 respectively and expected external damage is $0.38 per Bm^3 of excavation volume which in a project with estimated total direct cost of $1814359 may have impact about between 1% and 4% of estimated total direct cost however expected external damage is estimated at $49826 that impact is 2.75% of estimated total direct cost (illustrative example 9.7).

The sub-operations sharing cost on direct cost of deep excavation has been: excavation 25.46%, pile wall 37.21%, and anchorage 37.33% (illustrative example 9.7).

Influence of uncertainty conditions on job efficiency of clamshell or auger drill in retaining wall excavation is proposed in table 9.33 which includes weather condition, maneuvering, mechanical breakdowns, and operator efficiency in 16 situations and range from 0.61 until 0.833.

Optimistic and Pessimistic production, minimum duration, and rent cost for trenching of five case histories by hydraulic grab is estimated in illustrated example 9.9 which basically risk variables are grab fill factor, job efficiency, and swell factor. According to results because of geometry and volume of work, there is a significant difference among minimum duration and unit cost of trenching share in deep excavation cost due to direct rent per hour of clamshell and hydraulic grab which is shown in table 9.39.

Because of relatively near depth of wall trench for three first cases histories in illustrated example 9.9, the belonging production rates are nearly.

In situations such as example 9.9, the difference between optimistic estimated cost and pessimistic estimated cost is a risk cost potential interval due to situation. The mentioned deviation and impacts can be seen in unit cost of trenching share in deep excavation overall cost.

Range of unit cost of trenching share in deep excavation cost due to direct rent per hour of clamshell and hydraulic grab is differed depend on volume of work and trench depth which optimistic and pessimistic are \$0.11 and \$2.12 per Bm_3 in overall that is indicated in table 9.40.

Based on case histories, the potential risk of deep excavation in clay especially OL-CL, CH, and OL-CH with high level water table is very high and this type of works need to exactly the highest level of inspection and supervising. Also design checking before construction is crucial.

Design, implementing, and periodic controlling of the wall-wale connection of struts is important to mitigate risk.

Based on case histories, it is important for preventing any damages or problems before starting construction works the underground situation is studied.

One of the solutions to mitigate and deal the settlement and its effect on adjacent buildings damages is using underpinning which use micropiles and/or jet-grouting. Based on case histories the range of 54.4% to 56.5% of total induced settlement for underpinning, 24.7% to 34.4% of total induced settlement for diaphragm wall construction, 16.8% to 18.2% of total induced settlement for excavation in deep excavation cases in depth about 8 to 10 meters may used for initially risk assessment of adjacent building which selected to strengthening by underpinning.

Production rate and time of construction have impact on cost and scheduling as identified risks.

There is need to conditions had to be satisfied in deep excavation contracts which may be production rate, scheduling, transport on site, slurry tank and centrifugal pump coordination responsibility in clamshell procurement, responsibility of potential damage to the properties of third parties, responsibility of risk to the contractor’s labor force and equipment, responsibility of risk to partially completed works of the employer, and the other responsibilities, duties, and obligations of each of the client, the contractor, and the consulting engineer.

In deep excavation, loading of excavated soil is done by hydraulic backhoe commonly unless there isn't possible to construct temporary ramp into deep due to lack of location which needs to benching so that lower hydraulic backhoe excavate and dump the soil on upper level of bench and other hydraulic backhoe translate the soil to next upper level of bench to reach to ground level and load to dump truck.

In deep excavation, job efficiency of excavation can tolerate between 0.833 and 0.45 depends on situation if hauling is done by 100% efficiency.

There is necessary to state a penalty in contract of hauling contractor to observance of 100% using of payload which control is by weighing after movement and calculating deviation of filled and unfilled.

There is necessary to state a penalty in contract of hauling contractor to observance of minimum number of haulers so that the excavator is not waited for hauler.

Production of hauling is limited by road capacity which affects the travel time. A formula for estimating of road capacity is proposed which depends on stopping distance plus length of vehicles, and maximum speed. The estimated stopping distance is minimum distance while heavy traffics can surge the cycle time of loading.

The cost of geotechnical tests for a deep excavation case study in Northern Cyprus is less than 0.033% of direct cost of deep excavation operations while the tests is required for design of foundation of main building structure.

Site investigation and identification for geotechnical risk management is developed as:

1- Arrangement of Boreholes based on building code or accepted method
2- Boring and sampling so that:
   2-1- Test Soil grading and SPT in each layer
   2-2- If there is Water table, check and register
3- If clay is observed in a layer, test moisture content, shrinkage, liquid, and plastic limits for that layer
4- Parameters ranged for soils from expanded soil classification are used for deterministic sensitivity analysis.

Deterministic sensitivity analysis requires low and high quantity of parameters for calculating FOS that is determined from proposed classification for soils by comparing SPT-N and soil appearance which is determined by soil grading test.

Cost risk due to 50 days delay, caused by indirect cost without including other expenses such as rehabilitation expenses, debt installments, contractual delay penalty, depreciation, central office overhead residual, financial costs, and the cost of lost opportunity is estimated in 9.7.

Cost risk due to material additional consume, caused by increasing in conversion of wells diameter from 0.85 to 1.05 meter is estimated in 9.7.

Cost risk due to material price increase due to change in price of steel reinforcement bars from $400 to average $450 per ton is estimated in 9.7.

REFERENCES

[1] Do, T. N., Ou, C. Y., & Chen, R. P. (2016). A study of failure mechanisms of deep excavations in soft clay using the finite element method. *Computers and Geotechnics*, *73*, 153-163. http://www. Elsevier.com

[2] Gue, S. S., & Tan, Y. C. (1998). Design and construction considerations for deep excavation. *SSP Geotechnics Sdn Bhd*.

[3] Ergun, M. U. (2008). Deep Excavations. *The Electronic Journal of Geotechnical Engineering*, *Bouquet*, *8*. http://www.ejge.com/Bouquet08/UfukErgun/UfukErgun_ppr.pdf

[4] Tan, Y. C., & Chow, C. M. (2008, March). Design of retaining wall and support systems for deep basement construction–a Malaysian experience. In *Seminar on "Deep Excavation and Retaining Walls", Jointly orgnaised by IEM-HKIE, Malaysia* (Vol. 24). http://www.gnpgeo.com.my/download/publication/2008

[5] Kung, G. T., Hsiao, E. C., Schuster, M., & Juang, C. H. (2007). A neural network approach to estimating deflection of diaphragm walls caused by excavation in clays. *Computers and Geotechnics*, *34*(5), 385-396. http://www. Elsevier.com

[6] Kung, G. T. C., Ou, C. Y., & Juang, C. H. (2009). Modeling small-strain behavior of Taipei clays for finite element analysis of braced excavations. *Computers and Geotechnics*, *36*(1), 304-319. http://www.Elsevier.com

[7] Osman, A. S., & Bolton, M. D. (2004). A new design method for retaining walls in clay. *Canadian geotechnical journal*, *41*(3), 451-466., http://cgj.nrc.ca

[8] Hong, Y., Ng, C. W. W., Liu, G. B., & Liu, T. (2015). Three-dimensional deformation behaviour of a multi-propped excavation at a “greenfield” site at Shanghai soft clay. *Tunnelling and Underground Space Technology*, *45*, 249-259. http://www. Elsevier.com

[9] Nemati, K. M. (2007). Temporary structures, excavations and excavation supports. *University of Washington. Available from: courses. washington. edu/cm420/Lesson5. pdf [Accessed 23 December 2011]*.

[10] Land Transport Authority. (2004). Construction of contiguous bored pile wall, 1800-2255 582, http://www.lta.gov.sg

[11] Technical retaining supports figures (2015, December 9). Retrieved from http://www.skanska.co.uk

[12] Rawlings, C. G., Hellawell, E. E., & Kilkenny, W. M. (2000). *Grouting for ground engineering*.

[13] Castaldo, P., & De Iuliis, M. (2014). Effects of deep excavation on seismic vulnerability of existing reinforced concrete framed structures. *Soil Dynamics and Earthquake Engineering*, *64*, 102-112. http://www.Elsevier.com

[14] Paolo Gazzarrini. (2013). *Diaphragm wall: a solution for deep excavation in difficult soil conditions with superficial water table*, Sky Geotech Inc, West Vancouver, B.C., Canada

[15] Technical specification for hydraulic grab and clamshell (2016, May 14). Retrieved from http://www.casagrandegroup.com

[16] F.Harris. (1989). *Modern construction equipment and methods*, Longman, Singapore

[17] Technical specification for grab, clamshell, and excavator (2016, May 8). Retrieved from http://www.made-in-chaina.com

[18] Harris, F. (1983). *Ground engineering equipment and methods*. McGraw-Hill Companies.

[19] S.W.Nunnally. (2001) *Construction methods and management*- fifth edition- Prentice Hall

[20] Technical specification for hydraulic excavator (2016, January 24). Retrieved from http://www.cat.com

[21] Technical specification for hydraulic excavator (2016, January 25). Retrieved from http://www.deere.com

[22] Technical specification for hydraulic excavator (2016, January 26). Retrieved from http://www.komatsu.com

[23] Technical specification for excavator and dump truck (2016, January 28). Retrieved from http://www.Volvoce.com

[24] Handbook, C. P. (1998). Edition 29, Caterpillar. *Inc.*, *Peoria, IL*.

[25] Çelik Tahir. (2015). Construction methods and equipments course lecture note (CIVIL 596), Eastern Mediterranean University, Famagusta, Northern Cyprus

[26] Seo, J., Lee, S., Kim, J., & Kim, S. K. (2011). Task planner design for an automated excavation system. *Automation in Construction*, *20*(7), 954-966. http://www. Elsevier.com

[27] ASTM, D. 2487-06 (2008).". *Standard practice for classification of soils for engineering*.

[28] European Standard, EN 1537. (2000). *Execution of Special Geotechnical Work - Ground Anchors*

[29] European Standard, EN 14490. (2004). *Execution of Special Geotechnical Work - Soil Nails*

[30] Naresh Gurpersaud, Sai K. Vanapalli, Siva Sivathayalan. (2011). Pull-out capacity of soil nails in unsaturated soils, *Pan-Am CGS Geotechnical Conference.*

[31] Muntohar, A. S., & Liao, H. J. (2013). Finite Element Analysis of the Movement of the Tie-Back Wall in Alluvial-Silty Soils. *Procedia Engineering*, *54*, 176-187. http://www. Elsevier.com

[32] Technical specification for grout pumps (2016, June 11). Retrieved from http://www.grout.pumps.com

[33] Juang, C. H., Wang, L., Hsieh, H. S., & Atamturktur, S. (2014). Robust geotechnical design of braced excavations in clays. *Structural Safety*, *49*, 37-44. http://www. Elsevier.com

[34] Zhang, W., Goh, A. T., & Xuan, F. (2015). A simple prediction model for wall deflection caused by braced excavation in clays. *Computers and Geotechnics*, *63*, 67-72. http://www. Elsevier.com

[35] Ran, L., Ye, X. W., & Zhu, H. H. (2011). Long-term monitoring and safety evaluation of a metro station during deep excavation. *Procedia Engineering*, *14*, 785-792. http://www. Elsevier.com

[36] Sharma, J. S., Hefny, A. M., Zhao, J., & Chan, C. W. (2001). Effect of large excavation on deformation of adjacent MRT tunnels. *Tunnelling and Underground Space Technology*, *16*(2), 93-98. http://www. Elsevier.com

[37] Teparaksa, W. (2005). Performance of Contiguous Pile Wall for Deep Excavation on Chao Phraya River Bank. *Invited paper*.

[38] Katzenbach, R., Leppla, S., Ramm, H., Seip, M., & Kuttig, H. (2013). Design and construction of deep foundation systems and retaining structures in urban areas in difficult soil and groundwater conditions. *Procedia Engineering*, *57*, 540-548. http://www. Elsevier.com

[39] Lifson, M. W., & Shaifer, E. F. (1982). Decision and Risk Analysis for Construction Management. *JOHN WILEY & SONS, INC., 605 THIRD AVE., NEW YORK, NY 10158. 1982.*

[40] EN BS 6079-3:2000

[41] G & C. Merriam Co. (1965). *Webster's seventh new collegiate dictionary*. Springfield, Massachusetts

[42] Hammer, W., & Price, D. (2001). *Occupational safety management and engineering*. Pearson College Division.

[43] Raz, T., & Hillson, D. (2005). A comparative review of risk management standards. *Risk Management*, *7*(4), 53-66.

[44] Weihrich, H., & Koontz, H. (2005). *Management: A global perspective*. Tata McGraw-Hill.

[45] Office of Project Management Process Improvement. (2003). *Project Risk Management Handbook*, Sacramento

[46] Kremljak, Z., & Kafol, C. (2014). Types of risk in a system engineering environment and software tools for risk analysis. *Procedia Engineering*, *69*, 177-183. http://www. Elsevier.com

[47] Purdy, G. (2010). ISO 31000: 2009 — setting a new standard for risk management. *Risk analysis*, *30*(6), 881-886. http://www.iso.org/iso/home/standards/iso31000.htm

[48] Profession, A. Institution of Civil Engineers (2005). *Risk analysis and management for projects (RAMP), 2nd edition. Thomas Telford, London*.

[49] Project Management Institute. (2007). *Construction extension to the PMBOK® Guide*, 3rd ed. Newtown Square: Project Management Institute

[50] European Committee for Standardization. (2002). *EN 1990: Basis of Structural Design*

[51] Imriyas, K. (2009). An expert system for strategic control of accidents and insurers' risks in building construction projects. *Expert Systems with Applications*, *36*(2), 4021-4034. http://www. Elsevier.com

[52] Buchan, R. D., Fleming, E., & Grant, F. (2012). *Estimating for builders and surveyors*. Routledge, http://www. Elsevier.com

[53] Lind, D. A., Marchal, W. G., & Wathen, S. A. (2005). Statistical Techniques in Business and Economics. McGraw-Hill Companies, Inc.

[54] Aven, T. (2016). Risk assessment and risk management: Review of recent advances on their foundation. *European Journal of Operational Research*, *253*(1), 1-13. http://www. Elsevier.com

[55] Fewings, P. (2013). *Construction project management: An integrated approach*. Routledge.

[56] Smith, N. J., Merna, T., & Jobling, P. (2009). *Managing risk: in construction projects*. John Wiley & Sons.

[57] Bahrami, M., Bazzaz, D. H., & Sajjadi, S. M. (2012). Innovation and improvements in project implementation and management; using FMEA technique. *Procedia-Social and Behavioral Sciences*, *41*, 418-425. http://www. Elsevier.com

[58] EUROCODE 7, part 2, *Geotechnical design*

[59] Wudtke, R. B. (2008, September). Failure Mechanisms of Hydraulic Heave at Excavations. In *19th European young geotechnical engineers' conference, Gyor, Hungary*.

[60] Bond, A. J., Schuppener, B., Scarpelli, G., Orr, T. L., Dimova, S., Nikolova, B., & Pinto, A. V. (2013). Eurocode 7: Geotechnical Design Worked examples. In *Worked examples presented at the Workshop "Eurocode* (Vol. 7).

[61] A. J. Bond, G. Scarpelli. (2013). Retaining structures II, European commission JRC scientific and policy reports, Worked examples presented at the Workshop "Eurocode 7: *Geotechnical Design" Dublin, 13-14 June, 2013, Support to the implementation, harmonization and further development of the Eurocodes.*

[62] Bond, A. (2013). Implementation and evolution of Eurocode 7. *Modern Geotechnical design codes of practice: implementation, application, and development. Edited by P. Arnold, GA Fenton, MA Hicks, T. Schweckendiek, and B. Simpson. Advances in Soil Mechanics and Geotechnical Engineering, IOS Press, Amsterdam*, 3-14.

[63] Trevor L.L. ORR. (2002). *Implications of Eurocode 7 for Geotechnical Design in Ireland*, The institution of engineers of Ireland, Geotechnical Society of Ireland Society

[64] Çelik Tahir. (2016). Project Management course lecture note (CIVIL 694), Eastern Mediterranean University, Famagusta, Northern Cyprus

[65] Salam, M. A. (1995). Contractual sharing of risks in underground construction: ITA views. *Tunnelling and Underground Space Technology*, *10*(4), 433-437. http://www. Elsevier.com

[66] B. M Das. (2007). *Principles of Foundation Engineering*, Thomson India Edition, New Dehli

[67] Braja, M. D. (2008). Advanced soil mechanics. *Routledge Publication. ISBN*, *415420261*(9780415420266), 567.

[68] Morgan, C. O. (2003). *Excavation Safety: A Guide to OSHA Compliance and Injury Prevention*. Government Institutes. (About: OSHA, 1926 subpart P App A-Soil classification)

[69] Likitlersuang, S., Teachavorasinskun, S., Surarak, C., Oh, E., & Balasubramaniam, A. (2013). Small strain stiffness and stiffness degradation curve of Bangkok clays. *Soils and Foundations*, *53*(4), 498-509. http://www. Elsevier.com

[70] Cheng Liu, Jack B. Evett. (2000). *Soils and Foundations, Fifth edition, The University of North Carolina at Charlotte*

[71] Bowles, J. E. (1988). *Foundation analysis and design*

[72] Rogers, J. D. (2006). Subsurface exploration using the standard penetration test and the cone penetrometer test. *Environmental & Engineering Geoscience*, *12*(2), 161-179.

[73] Abdrabbo, F. M., & Gaaver, K. E. (2012). Applications of the observational method in deep foundations. *Alexandria Engineering Journal*, *51*(4), 269-279. http://www. Elsevier.com

[74] Shi, J., Liu, G., Huang, P., & Ng, C. W. W. (2015). Interaction between a large-scale triangular excavation and adjacent structures in Shanghai soft clay. *Tunnelling and Underground Space Technology*, *50*, 282-295. http://www. Elsevier.com

[75] Zheng, G., Zeng, C. F., Diao, Y., & Xue, X. L. (2014). Test and numerical research on wall deflections induced by pre-excavation dewatering. *Computers and Geotechnics*, *62*, 244-256. http://www. Elsevier.com

[76] Douglas C. Montgomery. (2013). *Design and Analysis of Experiments*, Eighth Edition, International student version, John Wiley & Sons Singapore Pte.Ltd.

[77] Woodcock, N. (1994). AC Waltham, 1994. Foundations of Engineering Geology. v+ 88 pp. London, Glasgow, New York, Toyko, Melbourne, Madras: Blackie Academic & Professional (Chapman & Hall). Price£ 9.95 (paperback). ISBN 0 7514 0071 8. *Geological Magazine*, *131*(06), 853-853.

[78] Moh, Z. C. (2004, August). Site investigation and geotechnical failures. In *Proceeding of International Conference on Structural and Foundation Failures* (pp. 2-4).

[79] Ministry of Interior. (2001). *The Building Design Code*, revised 2001, Taiwan

[80] American Standard Test Methods from: http://www.astm.com

[81] John E.Freund. (1992). *Mathematical statistics*, Fifth edition, Prentice Hall International Editions

[82] Bolton, M. D., Lam, S. Y., & Vardanega, P. J. (2010, May). Predicting and controlling ground movements around deep excavations. In *The 11th International Conference of the DFI-EFFC. London:[sn]* (pp. 26-28).

[83] D.G.Fredlund. (1996). *The Emergence of Unsaturated Soil Mechanics*, University of Saskatchewan, Canada

[84] Leong, E. C., Rahardjo, H., & Fredlund, D. G. (2001). Application of unsaturated soil mechanics in geotechnical engineering. In *Proceedings of the 8th East Asian Pacific Conference on Structural Engineering and Construction* (pp. 5-7).

[85] Kayadelen, C., Tekinsoy, M. A., & Taşkıran, T. (2007). Influence of matric suction on shear strength behavior of a residual clayey soil. *Environmental geology*, *53*(4), 891-901.

[86] British Standards Institution. (1994). *Code of Practice for Earth Retaining Structures (BS8002)*, British Standards Institution, London, UK

[87] European Standard, EN 14199:2005 *Execution of Special Geotechnical Work - Micropiles.*

[88] Prasad .S. K, Foundation Settlement, S. J. College of Engineering, Mysore, http://www.elearning.vtu.ac.in/10/enotes/06CV64/Unit8-SKP.pdf

[89] Bolton Seed, H., Tokimatsu, K., Harder, L. F., & Chung, R. M. (1985). Influence of SPT procedures in soil liquefaction resistance evaluations. *Journal of Geotechnical Engineering*, *111*(12), 1425-1445.

[90] Seed, H. B., & Idriss, I. M. (1971). Simplified procedure for evaluating soil liquefaction potential. *Journal of Soil Mechanics & Foundations Div*.

[91] Youd Tl. (1998). Liquefaction resistance of soils; summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. *J. Geotech Geoenviron Eng 2001*; 127(10):817–33.

[92] Iwasaki T, Tokida K, Tatsuoka F, Watanabe S, Yasuda S, Sato H. (1982). Microzonation for soil liquefaction potential using simplified methods. In: *Proceedings 3rd international conference on microzonation*, Seattle, USA; 1982. p. 1319–30.

[93] Iwasaki, T., Tatsuoka, F., Tokida, K. I., & Yasuda, S. (1978, November). A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan. In *Proc., 2nd Int. Conf. on Microzonation* (pp. 885-896). Washington, DC: National Science Foundation.

[94] Ivan B. Gratchev, Kyoji Sassa, Victor I. Osipov, Viatcheslav N. Sokolov. (2006). The liquefaction of clayey soils under cyclic loading, *Engineering Geology 86* (2006) 70–84. http://www. Elsevier.com

[95] Liyanapathirana, D. S., & Poulos, H. G. (2004). Assessment of soil liquefaction incorporating earthquake characteristics. *Soil Dynamics and Earthquake Engineering*, *24*(11), 867-875. http://www. Elsevier.com

[96] CEN European committee for standardization. (2004). EN 1998-5, EUROCODE 8: *Design of structures for earthquake resistance – part 5: foundations, retaining*

[97] Chung, J. W., & Rogers, J. D. (2010). Simplified method for spatial evaluation of liquefaction potential in the St. Louis area. *Journal of Geotechnical and Geoenvironmental Engineering*, *137*(5), 505-515.

[98] Iwasaki T, Arakawa T, Tokida KI. (1984). Simplified procedures for assessing soil liquefaction during earthquakes. *Soil Dyn Earthq Eng 1984*;3(1):49–58.

[99] Chung, C. K., Kim, H. S., & Sun, C. G. (2014). Real-time assessment framework of spatial liquefaction hazard in port areas considering site-specific seismic response. *Computers and Geotechnics*, *61*, 241-253. http://www. Elsevier.com

[100] Ibrahim, K. M. H. I., & Ibrahim, T. E. (2013). Effect of historical earthquakes on pre-stressed anchor tie back diaphragm wall and on near-by building. *HBRC Journal*, *9*(1), 60-67. http://www. Elsevier.com

[101] Elbadry, H. (2016). Simplified reliable prediction method for determining the volume change of expansive soils based on simply physical tests. *HBRC Journal*. http://www. Elsevier.com

[102] Slamet Widodo, Abdelazim Ibrahim. (2012). Estimation of Primary Compression Index (Cc) Using Physical Properties of Pontianak Soft Clay, Technische Universitat Bergakademie Freiberg, Germany, *International Journal of Engineering Research and Applications* (IJERA) ISSN: 2248-9622 Vol. 2, Issue 5, September- October 2012, pp.2232-2236, http://www.ijera.com

[103] Bartlett, S. F., & Lee, H. S. (2004). *Estimation of Compression Properties of Clayey Soils, Salt Lake Valley, Utah* (No. UT-04.28). Utah Department of Transportation.

[104] Yertek. (2015). Spring Mall sondaja dayali zemin ve temel etut raporu, Mühendislik araştirma ve sondaj san. Tic.Ltd.Sti, Tel:02122587967, http://www.yertek.com.tr

[105] Jan Niklas Franzius. (2003). Behavior of buildings due to tunnel induced subsidence, A thesis submitted to the University of London for the degree of

Doctor of Philosophy and for the Diploma of the Imperial College of Science, Technology and Medicine, London, UK

[106] Chen, S., & Xiang, Y. (2006). A procedure for theoretical estimation of dewatering-induced pile settlement. *Computers and Geotechnics*, *33*(4), 278-282. http://www. Elsevier.com

[107] Zhou, N., Vermeer, P. A., Lou, R., Tang, Y., & Jiang, S. (2010). Numerical simulation of deep foundation pit dewatering and optimization of controlling land subsidence. *Engineering geology*, *114*(3), 251-260. http://www. Elsevier.com

[108] Pujades, E., Vàzquez-Suñé, E., Carrera, J., & Jurado, A. (2014). Dewatering of a deep excavation undertaken in a layered soil. *Engineering geology*, *178*, 15-27. http://www. Elsevier.com

[109] Yertek. (2015). Spring Mall derin kazi iksa systemi geoteknik hesap raporu, Mühendislik araştirma ve sondaj san. Tic.Ltd.Sti, Tel:02122587967, http://www.yertek.com.tr

[110] Batu, V. (1998). *Aquifer Hydraulics: A Comprehensive Guide to Hydrogeologic Data Analysis*, John Wiley & Sons, New York, 727p

[111] Cui, Q. L., Shen, S. L., Xu, Y. S., Wu, H. N., & Yin, Z. Y. (2015). Mitigation of geohazards during deep excavations in karst regions with caverns: A case study. *Engineering Geology*, *195*, 16-27. http://www. Elsevier.com

[112] A. Abdolhosainzade. (1983). *Building estimating and cost* (in Persian language), Tehran, Iran

[113] The Management and Planning Organization of Iran, President Unity, (2008), *Analysis unit cost for road maintenance work* (in Persian language), Tehran, Iran, http://www.mporg.ir/Portal/View/Page.aspx?PageId=0c1adeb9-d144-46a3-9e5e-ced7504c23b3

[114] Project management institute. (2000). *A Guide to the Project Management Body of Knowledge (PMBOK Guide)*, Four Campus Boulevard, Newton square, Pennsylvania USA, http://www.pmi.org

[115] Chepurnova, A. (2014). Assessing the influence of jet-grouting underpinning on the nearby buildings. *Journal of Rock Mechanics and Geotechnical Engineering*, *6*(2), 105-112. http://www.Elsevier.com

[116] Federal emergency management agency, Department of homeland security. (2010). *FEMA's schedule of equipment rates*, Washington, D.C.

[117] Brake lag distance data. (2015, June 2). Retrieved from http://www.csgnetwork.com/stopdistcalc.html

[118] Brake lag distance data (2015, June 3). Retrieved from http://www.racemath.com

[119] Tire–road friction coefficient data. (2016, June 11). Retrieved from http://www.marules.com

[120] Kanwar Bharat Singh & Saied Taheri. (2015). Estimation of tire–road friction coefficient and its application in chassis control systems, Systems Science & Control Engineering, 3:1, 39-61, DOI: 10.1080/21642583.2014.985804