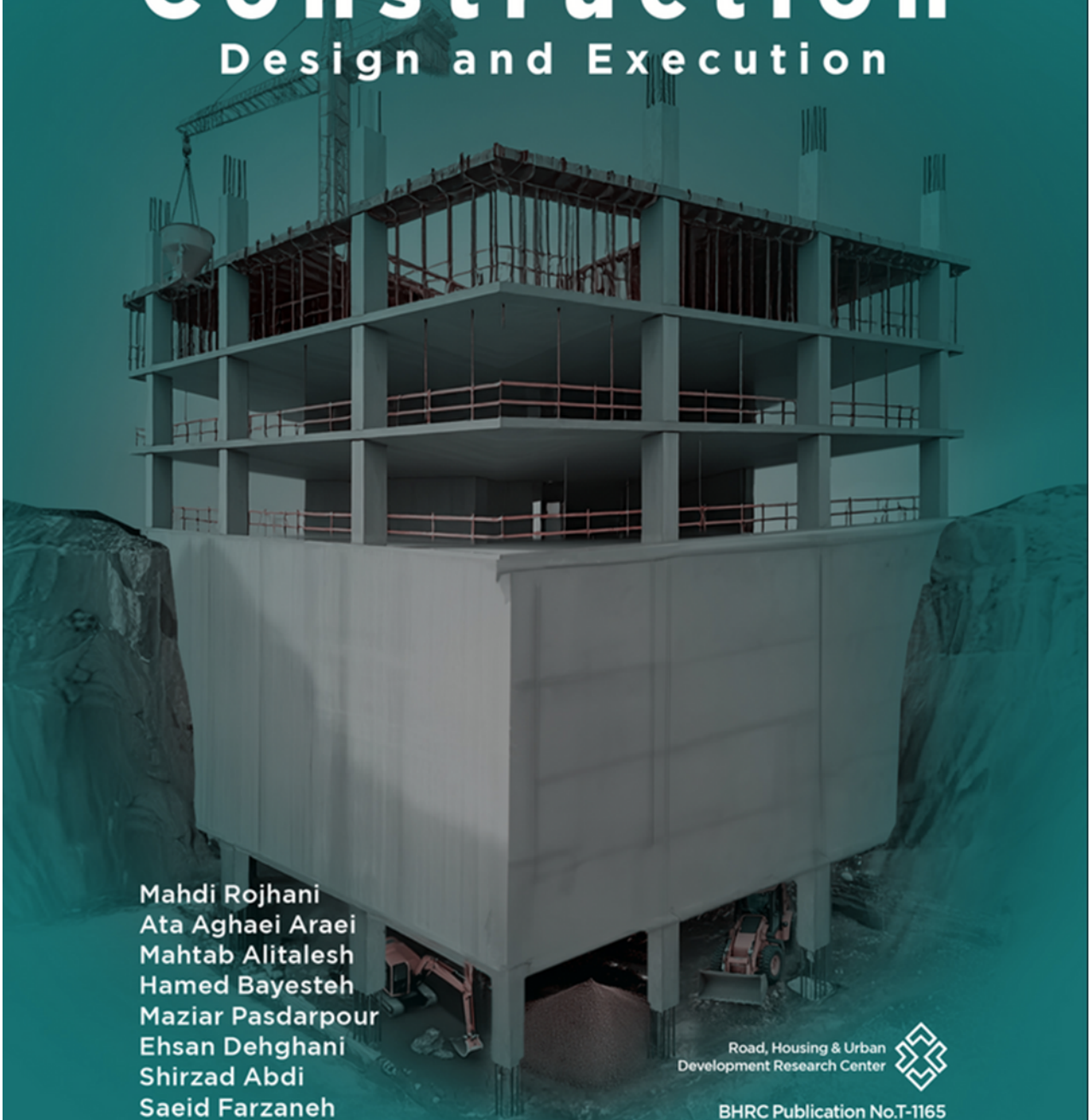
A Comprehensive Guide to Top-Down Construction

Design and Execution



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Road, Housing & Urban Development Research Center

BHRC Publication No.T-1165

In the name of Allah the most compassionate, ever compassionate

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Published: 2026

BHRC Publication No.T-1165

Ministry of Roads & Urban Development Islamic Republic of Iran

Road, Housing & Urban Development Research Center (BHRC)

سرشناسه	: A comprehensive guide to top-down construction design and execution[Book]
عنوان و نام پدیدآور	: تهران: مرکز تحقیقات راه، مسکن و شهرسازی، انتشارات، ۱۴۰۵= ۲۰۲۶م.
مشخصات نشر	: مرکز تحقیقات راه، مسکن و شهرسازی۱۴۰۵
مشخصات ظاهری	: xlviii، ۳۷۶ص.: مصور، جدول، نمودار
فروست	: مرکز تحقیقات راه، مسکن و شهرسازی، شماره نشر: ک-۱۱۶۵
شابک	: ۹۷۸-۶۰۰-۱۱۳-۵۸۲-۸
وضعیت فهرستنویسی	: فیپا
یادداشت	: زبان: انگلیسی.
یادداشت	: Authors Mahdi Rojhani, Ata Aghaei Araei, Mahtab Alitalesh, Hamed Bayesteh, Maziar Pasdarpour, Ehsan Dehghani, Shirzad Abdi, Saeid Farzaneh.
یادداشت	: کتابنامه:ص.۳۱۷-۳۲۷
موضوع	: کمپرهنسیو...
موضوع	: ساختمان‌سازی -- ایران
موضوع	: Building -- Iran
موضوع	: گودبرداری -- ایران
موضوع	: Excavation -- Iran:
موضوع	: گودبرداری -- ایران -- پیش‌بینی‌های ایمنی
موضوع	: Excavation -- Iran -- Safety measures:
موضوع	: ساختمان‌سازی زیرزمینی
موضوع	: Underground construction:
شناسه افزوده	: رجحانی، مهدی، ۱۳۶۰-
شناسه افزوده	: Rojhani, Mahdi, 1981-
شناسه افزوده	: مرکز تحقیقات راه، مسکن و شهرسازی
شناسه افزوده	: Road, Housing and Urban Development Research Center
رده بندی کنگره	: TH۱۴۵
رده بندی دیویی	: ۶۹۰/۰۲۹۵۵
شماره کتابشناسی ملی	: ۱۰۶۰۹۶۱۳
وضعیت رکورد	: فیپا

Road , Housing & Urban Development Research Center

Title: A Comprehensive Guide to Top-Down Construction Design and Execution
Authors: Mahdi Rojhani, Ata Aghaei Araei, Mahtab Alitalesh, Hamed Bayesteh, Maziar Pasdarpour, Ehsan Dehghani, Shirzad Abdi, Saeid Farzaneh
Publication No: B-1165
Publisher: Road, Housing & Urban Development Research Center (BHRC)
Edition: First Edition
Print Run: 200 Copies
Format: Royal Octavo (Vaziri)
Prepress, Printing, and Binding: Publications and Printing Department, Road, Housing & Urban Development Research Center (BHRC)
Price: ..؟؟؟.. Iranian Rials
ISBN: 978-600-113-582-8

The authors bear full responsibility for the accuracy and validity of the scientific views and opinions presented in this publication.
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Publisher Address: Hekmat Street, Shahid Ali Marvi Street, Sheikh Fazlollah Nouri Expressway, Tehran, Iran
P.O. Box: 13145-1696 **Tel**: +98 21 88255942-6 **Fax**: +98 21 88384132
E-mail: pub@bhrc.ac.ir
Online Bookstore: http://pub.bhrc.ac.ir

We dedicate this book to the memory of Iran's martyred architect of a New Iranian-Islamic Civilization, and to the martyred scientists of Iran—great souls and brilliant minds who were targeted not because of the weakness of their ideas, but because of the power of their vision and the light they brought to the world.

On June 13, 2025, and February 28, 2026, the illegitimate Zionist regime sought to extinguish the light of knowledge through cowardly and brutal attacks.

But blood shed in the path of truth does not vanish—it takes root, and it rises.

We also remember the innocent children of Gaza, and the 168 innocent children of Minab—whose laughter was buried beneath the rubble, whose dreams were shattered by war, and whose very existence is a cry against injustice.

This book stands as a vow—that we will carry the torch of science where they tried to sow silence; that we will learn, build, and rise—not in revenge, but in resistance; not with hatred, but with unshakable truth.

For knowledge cannot be killed, and the light of truth cannot be extinguished.

Publisher's Foreword

Environmental, economic, and social incentives have led to a prevalence of underground development in recent years and earthscrapers, also known as underground complexes, gradually replace skyscrapers in many parts of the world. In Iran, too, as the populations grow and cities develop, issues such as providing enough parking space, growing prices of land, and high-rise building limitations, leave developers with no option but to turn to underground alternatives. In the last two decades, urban excavations have become deeper and more numerous than ever and ill-advised attempts have led to lamentable results in many cases. Consequently, a systematic approach was adopted to regulate urban excavations in Iran, and new methods, including nailing and anchoring, were introduced to prevent further unfortunate incidents. However, safety and reliability concerns as well as legal implications of encroachment on a project's neighboring properties put an end to these methods' decade-long ubiquity in Iran.

In their attempts to find alternatives, developers have considered a wide range of approaches, chief among which are the top-down method, the strut method, and the soldier pile method. The top-down method is widely used in developed countries for soil retention in metro stations and other underground structures and excavations. It is safe as well as time- and cost-effective. Conventionally, using the top-down method involves the construction of a diaphragm wall. In Iran, however, where the soil is usually compacted and cemented, partition walls are rarely used. As such, developers have adjusted the top-down method in accordance with local needs. The apparent simplicity of the method, however, has led to high-risk execution attempts.

In response to the pressing need for a domestic method in compliance with technical principles, our center prioritized devising guidelines to provide developers and engineers with a sound technical template. We hope this guidebook can help unify current approaches, transfer local knowledge, mitigate risks associated with existing methods, and reduce legal issues inherent in excavations so

that the groundwork for the preparation of top-down design standards and regulations can be firmly established.

Ghazal Raheb

President of Road, Housing, and Urban Development Research Center (BHRC)

2026

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Translator's Foreword

This guide to top-down design and construction was developed in response to growing demand in Iran, coinciding with the method's rising popularity in the country's construction industry in spring 2023. During its preparation, extensive efforts were made to collect and draw upon all available global resources and publications related to top-down construction. However, the authors found no comprehensive or unified reference on the subject—only fragmented materials addressing specific subtopics. Following the successful release of the Persian edition, the Road, Housing, and Urban Development Research Center decided to translate and publish the guide in English, offering it as the first integrated and comprehensive resource on top-down building design and construction for engineers worldwide. It is hoped that this guide will serve as a valuable and practical reference for its readers.

The authors welcome all feedback, suggestions, and critiques of this work. Comments can be submitted to the following address:

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Finally, we extend our sincere gratitude to all those who contributed to the authorship and translation of this work, with special appreciation for *Mr. Pouyan Assar*'s invaluable assistance in translating the book into English

Mahdi Rojhani
Ata Aghaei Araei
Spring 2026

Author's Foreword

As populations grow and cities expand, our need for more residential spaces increases. Humans have always strived for overcoming their limitations and expanding their horizons. Buildings are getting higher and the number of their stories grows every day. As their heights increase, these buildings become subject to growing urban restrictions and regulations. In some cases, engineers have no choice but to turn to underground development to realize their vision. This approach is more common when it comes to parking spaces where sunlight, fresh air, and charming views are dispensable. In global development projects of recent years, however, underground levels and basement floors have been repurposed to meet new financial, technical, and environmental demands. So much so that underground complexes are gradually replacing skyscrapers. Similar trends are observable in Iran as well, especially in metropolitan areas, where legal necessities for procuring adequate parking space, in addition to the high value of land, have made projects with numerous basement levels—and consequently, excavations that can be up to seventy meters in some cases—prevalent.

The need for advanced technologies of deep excavation execution and stabilization is obvious. However, the need for these technologies rose much earlier than their availability. The country's engineering community set out to organize a response. The first excavation guidebook was first published by Tehran municipality in the 1990s. In the second half of 2000s, nailing and anchoring excavation methods became popular to meet the growing development demands. Despite revolutionizing the execution of deep urban excavations, these methods were not considered safe, reliable, or suitable for urban areas by many experts. A more serious concern, though, was the legal implications of these methods due to their encroachment on adjacent lots which put an end to their ubiquity by the early 2010s. In many cases, owners of adjacent lots either refused to authorize excavations or asked to be paid such huge amounts of compensation that these excavations were

virtually implausible. Accordingly, alternative approaches to excavation stabilization have become prevalent since. The top-down method is one of these approaches.

The top-down method is an inverse approach to structural execution. Unlike the conventional bottom-up approach to execution, in other words, in the top-down method, the construction of buildings with basement levels begins at the ground level and continues down towards the building's foundation. In some cases, superstructure floors are built simultaneously. The main elements of the structure are built before excavation, and they must retain the excavation site walls as well. Essentially, the temporary wall retention structure is done away with and, similar to the utilization phase, the main structure is tasked with soil retention. The building of the structure and the excavation are carried out simultaneously on the project's site; therefore, the execution phase can be more sophisticated. This phase should be carried out gradually and carefully with utmost caution and every engineering and technical precaution taken into account.

The top-down method has been prevalent in developed countries for constructing underground structures. Engineers have used the method to build metro stations and other structures requiring excavation. According to technical literature, this method is considered safe, cost-effective, and time-effective. In Iran adjustments had to be made to the method before it could be used effectively. Usually, construction using the top-down method involves a peripheral partition wall. In Iran, however, due to various considerations—including the compacted, cemented soil of Tehran where many notable projects are located, the high cost of execution, the high value of land, urban restrictions, land lot restrictions, and excavation equipment disproportionality—there is no integrated partition wall. Given the local particularities, the top-down method had to be modified first. The variety of projects under construction, though, meant that domesticized variations of the method were erratic. Some of these variations comprise creative solutions, while others overlook essential technical considerations. The deceptive simplicity of the top-down method spawned unsafe variations. On the other hand, the many experiences gained during

the execution of these projects created a rich reserve of implicit knowledge in Iran which has remained unpublished for the most part, out of the engineers' and experts' reach. Technical, financial, and especially legal necessities have made the method popular while negligence of its principles and vital finer points as well as its capacities and limitations are more widespread than ever.

This real, urgent need in the construction industry for clear guidelines meant that a comprehensive set of directions based on technical principles had to be prepared to organize the existing explicit knowledge and enlist experts' support to collect the growing implicit knowledge. Only then a model designed for Iranian construction needs in accordance with global standards could be devised.

This guidebook of top-down method, the first of its kind, is the result of a collaboration between a group of experts currently active in the Iranian construction industry. The first sparks of its creation came into being in the summer of 2017 at the geotechnical and infrastructure office of Iran's Road, Housing, and Urban Development Research Center. It was later approved at an official meeting of a panel of experts at the same office. It should be noted that, as of the time of writing, there are no comparable guidebooks on the topic published in the world, making the present effort the first of its kind to attempt a comprehensive set of design and execution principles. The authors hope their attempt can help advance engineers' knowledge both in Iran and all around the world.

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Spring 2023

Acknowledgments

The authors would like to extend their gratitude to all the experts who helped them prepare the present book. First and foremost, our colleagues at Road, Housing, and Urban Development Research Center, including Ghazal Raheb, the president; Mohammad Mahdi Heydari; Saeed Bakhtiari, deputy director of research and technology; and Taha Tabatabaei Aghda, head of the geotechnical and infrastructure office who made this project possible. We would also like to thank Nahid Attarchian, the former head of the geotechnical and infrastructure office; Abolghasem Moezi; and Zakaria Vaeizi who helped us through the process.

The honorable board of review, Professor Ali Fakher, Professor Abbas Ghalandarzadeh, Professor Farzin Kalantari, and Professor Rasoul Mirghaderi reviewed the text and provided us with corrections and suggestions for improvement to make the guidebook more useful and effective.

Finally, we would like to express our sincere gratitude to members of the editorial team, who, working collaboratively, undertook the arduous task of compiling the materials and preparing the text. They are, in alphabetical order: Shirzad Abdi, Mahtab Alitalesh, Hamed Bayesteh, Ehsan Dehghani, Saeed Farzaneh, and Maziar Pasdarpour. Furthermore, Mahdi Shahidikhah helped us proofread the text and Hossein Ali Rahimi and Matin Safari Nasab helped us prepare it for publication.

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Spring 2026

Abstract

Environmental, financial, and social considerations have made underground spaces more popular all around the world, so much so that earthscrapers, or underground complexes, are gradually replacing skyscrapers. In Iran, too, as populations grow and cities expand, new challenges—such as parking space procurement, surging land prices, and high-rise building restrictions—have emerged that make using underground spaces all but inevitable. In the last two decades, urban excavations have become deeper and the number of extensive excavations has grown considerably. As the number and variety of unfortunate excavation-related accidents grew in the 1990s, several guidelines were published to address the issue. In the second half of the 2000s, as construction engineers' needs grew, nailing and anchoring excavation methods emerged as popular choices. On top of their safety and reliability challenges, however, these methods entailed serious legal issues due to their property rights infringements. As a result, their widespread use declined after a decade of undisputed ubiquity.

In their attempts to find an alternative, engineers experimented with the top-down method. Usually, excavation using the top-down method involves a diaphragm wall. However, due to the compacted, cemented nature of the soil in Tehran—where most notable projects are located—this element is absent in most excavation sites. As such, to make this method work in Iran engineers had to adjust it according to local needs. But the method's deceptive simplicity meant that many of these adjustments led to unsafe execution practices.

The present book attempts to provide clear guidelines for a domesticized version of the top-down method designed to meet the specific needs of the Iranian construction industry. We have included a precise definition of the top-down method, along with its requirements and specific features. Both the global and the local variations of the method have been described and their differences, the advantages and disadvantages of each, and their plausibility particulars have been outlined. Furthermore, the equipment,

facilities, and tools required by the method have been specified in detail. A comprehensive description of the structural design and excavation principles has also been provided. Additional considerations, including managerial concerns, risk management, monitoring, mapping, facilities, and other relevant factors have also been addressed. In preparing this guidebook, our goal has been to provide structural engineers, geotechnical engineers, construction engineers, and site managers worldwide with comprehensive guidelines for carrying out a successful project using the top-down method.

Keywords: excavation, top-down method, top-down, underground structure, design and construction guide

Chapter 1

An Overview of the Top-Down Method

Chapter 1: An Overview of the Top-Down Method

1.1 Introduction

This guidebook incorporates a set of design and construction principles for the top-down method. As the top-down method gains popularity in urban excavation retention, the lack of a guidebook dedicated to its proper execution is keenly felt. The common variation of the method in Iran deviates considerably from the global standard. In the following chapters, we will present a precise, scientifically-grounded picture of this variation to help engineers worldwide benefit from insights their Iranian peers have gained through years of experience using the method. The first chapter includes an outline of this guidebook's aim and range of applicability as well as an overview, where we provide a definition of the top-down method as well its background, purpose, relevant building codes, and the place of creativity in its execution.

Using the top-down method involves many calculation and construction intricacies, and its numerous variables can differ significantly depending on any specific project. The challenges of construction using this method is similar to underground and tunnel construction in that its plethora of technical, construction, and equipment-related considerations are closely intertwined. We hope to help engineers address these challenges through the information provided in this guidebook.

1.2 Aim

We aim to provide readers with a guidebook of design and construction using the top-down method. We also attempt to gather the documented knowledge on this subject, record the implicit knowledge produced in Iran and worldwide, articulate them as a coherent set of ideas, and finally, present them as a set of productive guidelines for designers, constructors, and supervisors. Furthermore, we will compare the Iranian variation of the top-down

method with the standard variation common worldwide, and point out the advantages and disadvantages of each approach.

1.3 Range of Applicability

We developed this guideline with the goal of practicality for designers, constructors, and supervisors. We considered all technical elements and their intricacies to provide the reader with a comprehensive set of guidelines, ensuring the projects devised using this guidebook are safe and of a high quality.

Designer engineers can use this guidebook to make their calculations and evaluate their top-down models. The top-down method entails multiple construction stages and, compared to some other methods, can require more vigorous analysis, design, and evaluation checks before it can be implemented safely.

Construction authorities that evaluate projects and issue licenses can use this guidebook as well. In recent years, many projects have been designed using this method and submitted for review while a comprehensive blueprint for evaluation remains lacking.

This guide outlines the criteria for evaluating the feasibility of using the top-down method, selecting the optimal approach based on project conditions, understanding the impacts of different top-down techniques on architectural design and structural stability, explaining loading conditions and changes in lateral earth pressure during underground construction, and establishing design standards and control measures. It also covers execution details, safety considerations, legal, environmental, and managerial aspects, equipment requirements, and necessary utility considerations for the successful implementation of the top-down method.

1.4 The Top-down Method: An Introduction

There are many methods of deep excavation stabilization, including nailing, anchoring, truss retaining, and strut retaining. Each method comes with a set of specific advantages and disadvantages. Deep excavation entails significant changes in the stress fields of the

adjacent structures and causes considerable deformations in them. Choosing the right excavation method and managing deformations—especially when there are other buildings next to the construction site—is an important task. In general, two main variables are used to determine the impact of excavation on adjacent structures: ground settlement and wall deformation. These two variables must be evaluated continuously during excavation operations. A review of different excavation methods has demonstrated that the top-down method is the most successful in soil stabilization (Long, 2001) and as an effective method of deformation management in deep excavations, the top-down method is the recommended approach for urban projects.

In the top-down method, the main structure itself acts as a permanent support system, eliminating or significantly reducing the need for temporary retention structures. This approach has earned the moniker 'structure as permanent strut' (SPS). In this method, the structure is subjected to active and at-rest soil pressures on one side, while passive resistance is mobilized on the opposite side. Alternatively, active pressure can be applied to both sides to ensure overall structural stability.

Construction stages for an underground structure using the top-down method include construction of columns, excavation of the first basement floor, construction of the peripheral wall, and construction of the first basement floor slab. The cycle of soil excavation, construction of peripheral walls, and construction of the next basement floor slab continues until the foundation level is reached and the main foundation of the building is constructed. A schematic of a conventional top-down construction procedure with three basement floors is presented in Figure 1.1.

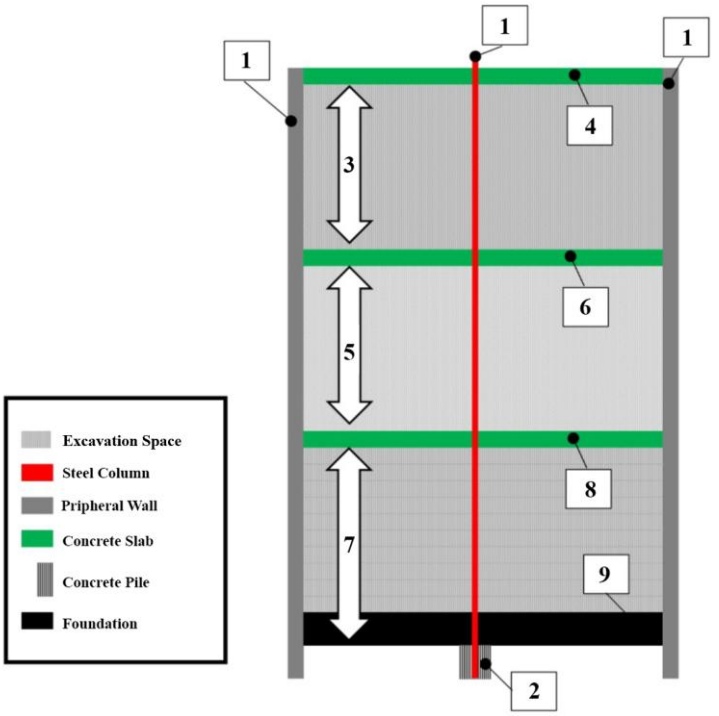


Figure 1.1 Top-down method construction stages

If the integrated peripheral wall is constructed as a diaphragm wall before excavation, there is virtually no local instability in the soil. Furthermore, the stiffness of the main structural floor slabs controls the deformation of the peripheral wall, and the peripheral retaining wall bears the gravity load of the main structure.

Key characteristics of the top-down method:

- It makes the temporary retention structures unnecessary as the main structure is used to stabilize the excavation during construction stages, and it allows for the simultaneous construction of superstructure and substructure as well as phase commissioning, both of which help save costs, an important cost management consideration.

- It can be used in cases where legal restrictions related to adjacent land lots, or time and workspace limitations make other methods impracticable.
- It requires no temporary retention structure which saves materials–such as steel and cement–that would be otherwise needed to stabilize the excavation site. This, on a large scale, would save energy and produce less greenhouse gasses, making the top-down method a green approach in accordance with sustainable development considerations.

The top-down method is an inherently safe approach where soil-structure interactions and peripheral walls' considerable friction forces bolster the structure's resistance under gravity and peripheral loads even further. While these elements are always present, in the top-down method, their role in the overall stabilization of the structure and its safety is more prominent, as they make the structure more stable under lateral load from soil pressure (Figure 1.2).

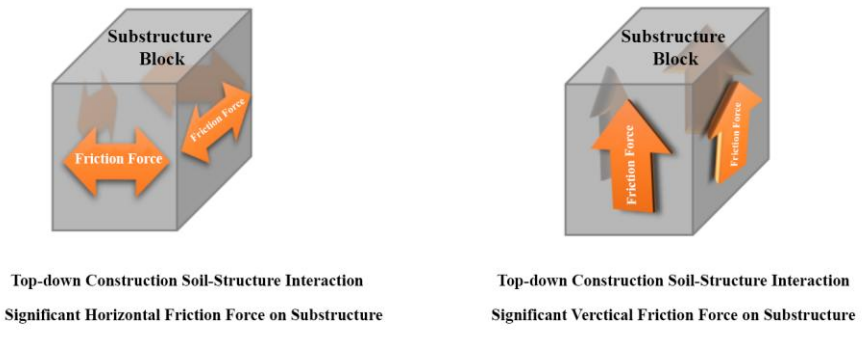


Figure 1.2 Frictional interactions in a substructure block

The efficiency of the top-down method hinges on the successful simultaneous execution of construction and excavation operations. Choosing the best access route to underground sites as well as the right excavation and removal method and equipment is crucial to operational success, especially in terms of time and cost management. From a project management perspective, this method

expedites construction by removing excavation from the critical path, enhancing project time management. The top-down method is a dynamic, innovative approach, providing many opportunities for optimization through creativity.

In technical literature, the top-down method is referred to by many monikers, including top (up) to down (bottom) construction, top (up)-down (bottom) construction, reverse (downward) construction, and superstructure-substructure construction.

Some notable projects that have benefited from the top-down method are Xingye Bank headquarters in Shanghai, Galleries Shopping Center in London, Alfa-Arbat Center in Moscow, Festival Walk in Hong Kong, International Finance Center in Hong Kong, Bitexco Financial Center in Ho Chi Minh City, Sky City Tower in Hanoi, Royal City in Hanoi, and Buona Vista MRT Station in Singapore. One of these projects, the International Finance Center in Hong Kong is a mixed-use development comprising a hotel and commercial office space. Located on the Hong Kong harbor, this iconic skyscraper is the world's third tallest building. Construction began in 1996 and was completed in 1998.

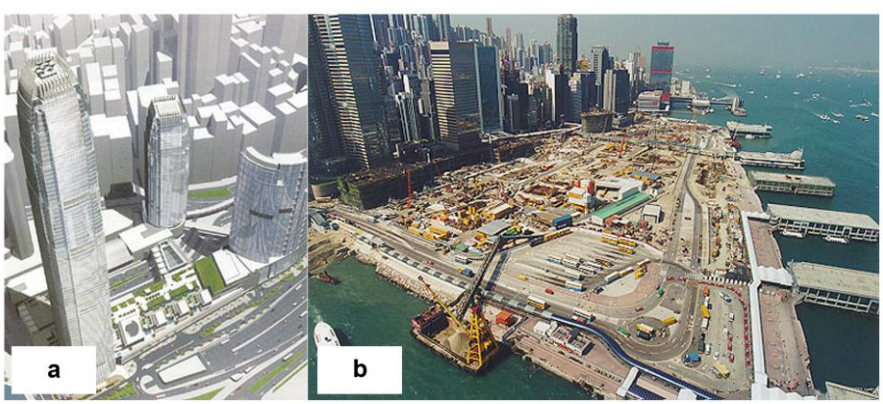


Figure 1.3 a) A graphical representation of the International Financial Center and its full development plan; b) The bird's eye view of the building.



Figure 1.4 Basement level excavation under the floor slab. Soil removal trucks are on the ground level next to soil elevators

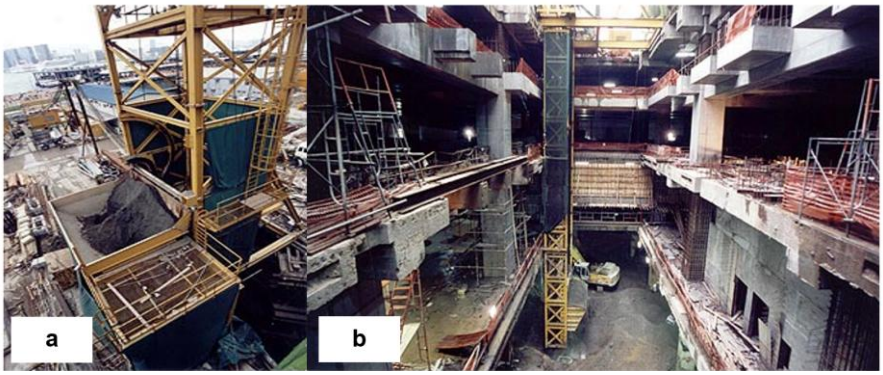


Figure 1.5 a) A crane transfers the excavated soil through an opening; b) Ongoing excavation at level -3 under the lowest floor slab, where excavated soil is deposited in the center to be transferred by the crane.



Figure 1.6 Ongoing excavation under a constructed slab while the ceiling and columns of the upper floor are already in place.

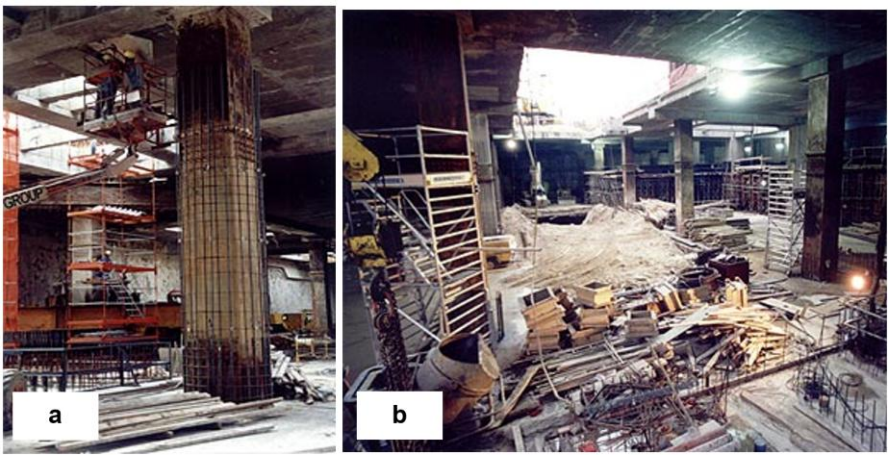


Figure 1.7 a) ongoing top-down process in basement floors. Notice the steel piles that were installed before excavation began to support ceilings; b) Steel piles are enclosed in reinforced concrete and are used as the final column of the basement floor.

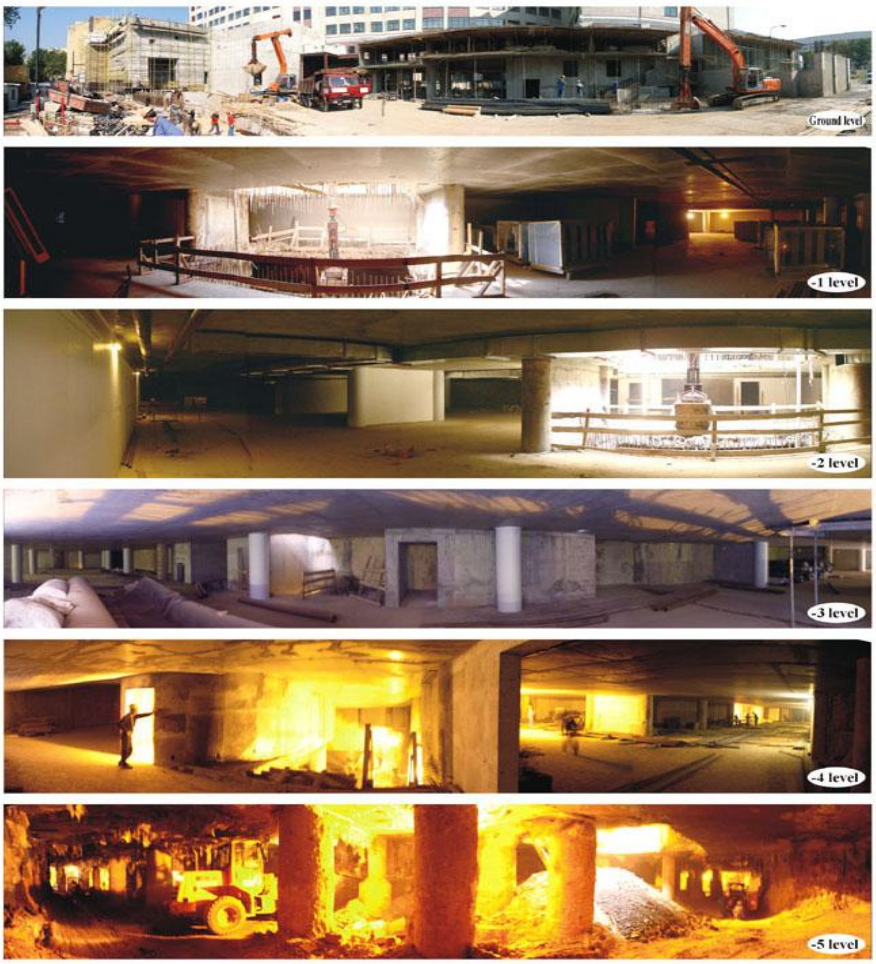


Figure 1.8 Alfa-Arbat Center in Moscow with five basement floors. Simultaneous operations in different floors.

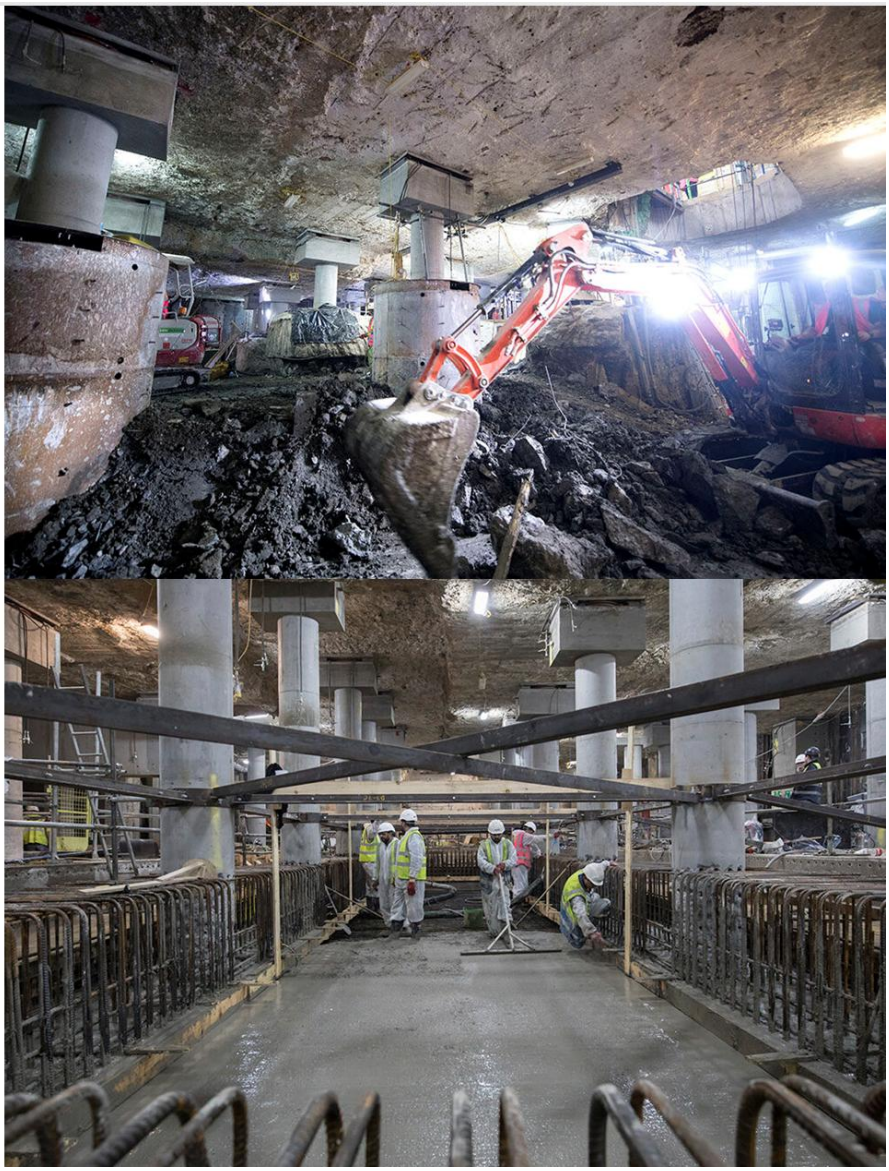


Figure 1.9 Claridge's Hotel development in London. Construction of five underground levels with an excavation depth of ~22 m, while the historical building is preserved

1.5 A History of the Top-down Method

1.5.1 The Conventional Top-down Method

Construction using the conventional top-down method is carried out following the process demonstrated in Figure 1.1. As documented in many case studies and papers, in the conventional method, the process always begins with the construction of the integrated peripheral wall using diaphragm walls, secant piles, or sheet piles.

The top-down concept, introduced in Japan in 1935, was developed to reduce construction time. Fifteen years later, in 1950, Italian engineers in Milan incorporated the diaphragm walls into the method to carry out excavation below an underground water table. Since then, the top-down method has been used for constructing metro stations in Paris and Milan to reduce operational costs and avoid traffic disruptions. Its success led to widespread adoption and it all but replaced traditional methods in industrial countries. Today, it is popular as a method of building skyscrapers with deep underground floors in areas with soft soil conditions where shoring is required to prevent soil wall collapse.

The method's applicability extends beyond new construction. It also enables the expansion of historical buildings while preserving their existing structure. By adding basement levels, this technique allows for significant growth without compromising the original architecture. In the United Kingdom, in particular, many historical buildings have been expanded using this approach.

1.5.2 The Development and Application of the Top-down Method in Iran

The top-down method was first used in Iran for infrastructure bridge and tunnel projects. With the expansion of the metro network in the 1990s, the top-down method emerged as a preferred choice for constructing multi-level underground structures. In all these cases, the conventional variation of the method was used. However, in the early 2000s, as legal challenges hindered the use of typical deep excavation stabilization methods like nailing and anchoring, the top-down method emerged as an increasingly

favored alternative in building construction. Using the top-down method has not always been in accordance with the conventional model widely adopted worldwide and Iranian engineers have refined it in many ways. They have adjusted it to fit local needs and adapted it to make it appropriate for medium-sized urban projects, tailored specifically for the Iranian context.

The Iranian variation of the top-down method comes with an important difference to the conventional one. As demonstrated in Figure 1.10, in the Iranian model, the first phase of the conventional technique, the construction of the integrated peripheral wall, is replaced with a sequential peripheral wall (SPW) approach. As demonstrated in Figure 1.10, each top-down project comprises two main parts: the top-down construction of the substructure and the bottom-up construction of the superstructure. According to the project manager's discretion, the construction of the substructure and superstructure may or may not progress simultaneously.

The substructure construction differs significantly between the two methods. In the conventional model, the initial phase involves constructing the integrated peripheral wall, middle columns, and ground floor slabs. The sequential phase then entails soil excavation and substructure floor slab construction. Finally, the main foundation is built.

In contrast, the SPW approach begins with drilling a shaft, constructing surrounding and middle columns with temporary foundations, and building the first basement floor slab. The sequential phase involves soil excavation and the iterative construction of substructure walls and slabs. The final phase remains the same: construction of the main foundation.

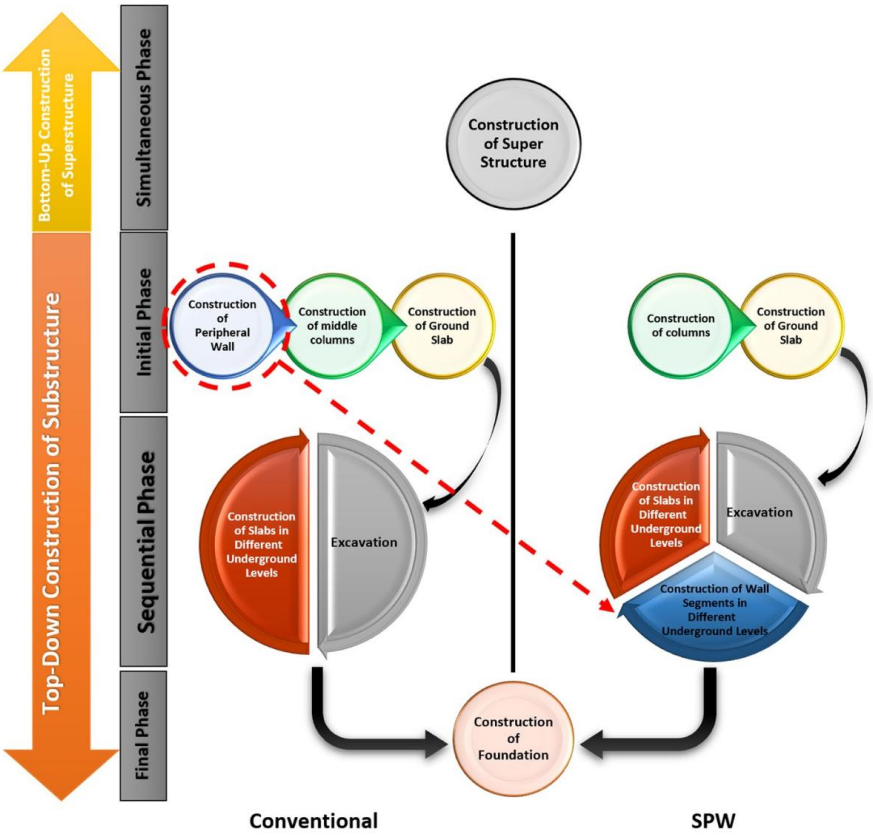


Figure 1.10 A schematic representation of the top-down method: the conventional approach versus the SPW approach

In addition to changing the construction process, the adjustments of the SPW approach entail structural and soil behavior changes that call for new design considerations. Unlike the conventional approach, the SPW approach lacks excavation protection, leaving the soil between columns exposed throughout extended phases of excavation, and mandating reliance on the soil's self-stability and stand-up time. Figure 1.11 compares the conventional and the SPW approaches in a structure with three basement floors.

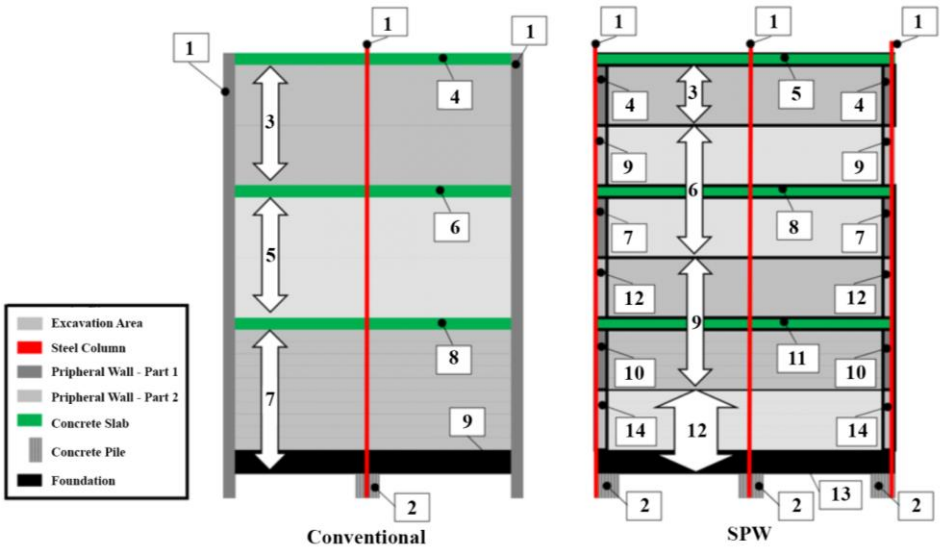


Figure 1.11 Construction stages in the conventional approach compared to the SPW approach

In the absence of a peripheral wall, the observational method (OM) should be adopted during design and execution to monitor wall stability throughout excavation operations. Modeling predictions must be constantly verified against field data to inform timely adjustments to excavation and construction procedures.

Most top-down projects adopting the SPW approach have been carried out in Tehran where the geotechnical soil conditions are favorable. The conventional method should not be written off completely, as in the relatively unfavorable geotechnical conditions in some parts of Iran, a diaphragm wall is a necessity. However, engineers can adopt different techniques to implement it, including in-situ concrete slabs and piles.

1.6 Notable Iranian Example Projects

In recent years, many construction projects with different structural designs have been carried out using the top-down method in Iran. This section presents some of these projects, most of which have adopted the SPW approach, alongside examples of the conventional

approach. Some examples of using the SPW approach include steel structures with waffle slabs or slab-and-beam designs; concrete structures with precast and semi-precast slabs; concrete structures with in-situ casting; as well as structures using composite sections such as Concrete Filled Tube (CFT) or Steel Reinforced Concrete (SRC).



Figure 1.12 Yousef Abad project, SPW approach: CFT, waffle ceilings, semi-precast slabs, retaining wall as an ancillary element, an excavation depth of 17 meters



Figure 1.13 Niayesh multistorey parking garage, SPW approach: concrete structure with precast columns, an excavation depth of 35 meters



Figure 1.14 Shariati-Falsafi project, SPW approach: SRC, an excavation depth of 34 meters



Figure 1.15 Andarzgoo project, SPW approach: SRC, an excavation depth of 18 meters



Figure 1.16 Pasdaran project, SPW approach: steel structure, steel deck ceilings, an excavation depth of 18 meters



Figure 1.17 South Dibaji project, SPW approach: concrete structure, in-situ columns, an excavation depth of 14 meters



Figure 1.18 Shahid Kolahdouz project, SPW approach: steel structure, waffle slab underground ceilings, an excavation depth of 22 meters

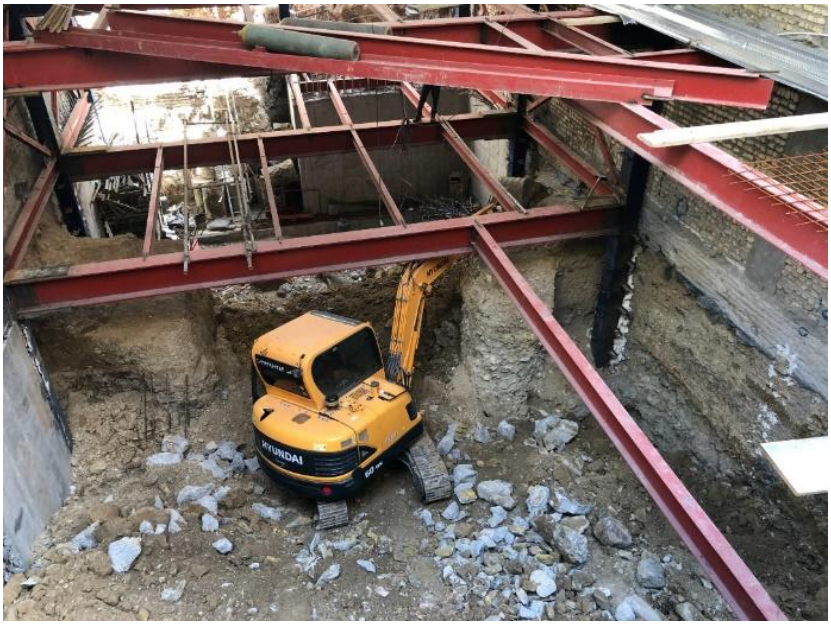


Figure 1.19 Elahiyeh project, SPW approach: steel structure, steel deck ceilings, an excavation depth of 7 meters in a 100 m2 lot



Figure 1.20 Sadeghiyeh project, SPW approach: steel structure, steel deck ceilings, one-meter-wide buried semi-precast concrete slabs as retaining walls, an excavation depth of 7 meters in an 80 m2 lot

1.7 Main Principles of the Top-down Method and its Purpose

The main purpose of the top-down method is to use the main structure to stabilize excavation walls. The main structure's stabilizing properties, then, are not limited to the post-construction period, and it should support the soil even during construction, eliminating the need for temporary retaining structures.

In the top-down method, the structure's high rigidity reduces deformations around the excavation shaft as well. Extensive field measurements and numerical analyses have demonstrated that floor slabs constructed using this method are rigid enough to control deformations in excavation walls as well as ground settlement in surrounding areas. As such, the top-down method is considered one of the safest methods of excavation and construction in high-density urban areas.

1.8 Creativity and Innovation in the Top-down Method

Given the intricacies of soil-structure interactions and the nuances of underground construction, engineers must use innovation to optimize these operations. Experts have explored many creative solutions to improve the top-down approach. Researchers like Lee et al. (1999) and Hong et al. (2010) proposed refinements to top-down principles, while others, such as Wang et al. (2010), focused on improving excavation efficiency. Zhou (2010) proposed more substantial changes, introducing the island technique, a combination of top-down and bottom-up methods. Numerous 3D numerical studies have been conducted as well to compare the top-down method against the bottom-up method.

These studies have all strived to understand, refine, and optimize the top-down method. As mentioned before, the top-down method is an innovative and creative approach by nature. The SPW variation of the top-down method, too, is a product of innovation on the Iranian engineers' part to address the high costs of constructing integrated diaphragm walls in compact, cemented soil conditions where these same properties can be utilized as stabilizers.

Other innovations in the top-down method include shaft head leveling devices to install suspended columns; using suitable connections and platforms for plunge column installation within a shaft; using suspended formworks to carry out excavation and concreting operation simultaneously and enable concrete curing in slabs at the same time; and using semi-precast concrete slabs to reduce material weights and enable easier installation compared to using fully-precast concrete slabs. Similar creative solutions are proposed every day. With this guidebook, we aim to establish essential principles of the top-down method to enable and facilitate these innovations.

1.9 The Top-down Method in Construction Codes

The top-down method is absent from most codes, regulations, and standards, with only England's BS:8002 2015 referring to its use as an earth retaining method.

Chapter 2

Principles and Feasibility

Chapter 2: Principles and Feasibility

2.1 Introduction

In the previous chapter, we provided a history of the top-down method and its development, compared conventional and SPW approaches, and presented example projects to offer an overview of the top-down method. Further, we pointed out the characteristics that make the top-down method a fertile ground for innovation.

In this chapter, we introduce the main principles of the top-down method, explore its strengths and weaknesses, and discuss its feasibility criteria. Specifically, we will determine how structural type and construction method can impact the top-down process.

By the end of this chapter, readers should have a solid picture of the top-down method. Subsequent chapters will be dedicated to practical intricacies of its application.

2.2 Principles

As demonstrated earlier, the top-down method comprises multiple procedures, each with specific prerequisites. To fully understand the technical intricacies and potential pitfalls of the top-down method, it is crucial to grasp its fundamental principles.

2.2.1 Main Structure as the Retaining Element

In most construction projects, the main structure is the retaining element in the post-construction period. While the construction is still under way, however, a temporary retaining structure is needed to maintain soil wall stability. This retaining effect can be achieved through construction of trusses or struts, or through nailing and anchoring systems. A core principle of the top-down method is that the main structure acts as the retaining element for

the soil wall during the construction as well as in the post-construction period.

2.2.2 Sequential Construction

As the name implies, the top-down method involves simultaneous construction of both above-ground and below-ground elements, starting from the ground level and progressing downwards as excavation occurs. To achieve this, technical and procedural adjustments are necessary. Furthermore, the main structure must be designed to withstand a variety of loads whose type is dictated by construction conditions in different stages. In other words, as the structure is being constructed, it must withstand soil loads as well as structural dead and live loads, a fact that should be considered in both design and execution phases.

2.2.3 Structural Stability and Deformation Control

The load-bearing capacity and stiffness of a structure are reduced while it is under construction, and its behavior can be unpredictable under temporary loads of construction operations. An under-construction structure might exceed its limits under much lower loads and become subject to prohibitive stress forces and deformations which can lead to structural failure. Therefore, the structure's capabilities must be monitored carefully in each construction stage. The complexities of determining and preventing critical conditions make the top-down method a challenging approach when it comes to structural design. The designer must have a firm grasp on both theoretical principles and operational considerations.

2.2.4 Underground Operations

A significant portion of a top-down project is carried out below the ground level, necessitating specialized techniques and considerations similar to other underground projects.

One of the main challenges of underground operations is the limited space. In these conditions, operational cycles must be planned carefully to minimize disruptions. Additionally, fresh air

and natural light are scarce in underground spaces and, especially at greater depths, this scarcity can impact safety and performance.

To mitigate these challenges, underground operations may call for specialized techniques, including efficient soil removal and transportation systems. Moreover, advanced surveying methods are necessary to accurately monitor underground conditions and operational progress.

Another restriction imposed by underground conditions is that engineers and experts cannot access all areas of the site, meaning operations are carried out by construction workers in absence of direct supervision. In these circumstances, simple but detailed instructions should be provided to minimize the necessity for direct supervision. Creative, technology-based solutions can also be used to ensure quality and safety.

Given the complex nature of underground construction, more lenient tolerances may be necessary compared to above-ground projects.

2.2.5 Underground Tools and Equipment

Significant progress has been made in developing specialized tools and equipment for underground operations. By understanding and efficiently using these tools, engineers can use them more efficiently. Excavation, soil removal, and soil transfer operations can all benefit from using tools and equipment specifically designed for underground sites.

2.2.6 Importance of Project Management

Considerations specific to the top-down method can make its application more complex compared to other methods, calling for meticulous project management. Numerous challenges, including constant disruptions, tight timelines, intricate interfaces, heightened risks, and execution difficulties make effective management indispensable in top-down projects. A comprehensive, systemic approach is essential to achieve perfect harmony between all project components.

2.3 Strengths

The top-down construction method offers a range of advantages that contribute to its appeal in complex projects, particularly in urban settings. From safety and cost efficiency to environmental benefits, these strengths make the top-down approach a viable and often preferable choice for various construction needs.

2.3.1 Functionality

The top-down method offers significant financial and social advantages by enabling the early construction of ground and above-ground floors. This allows for the stage-by-stage utilization of building sections, facilitating early project marketing and presale campaigns, which are particularly beneficial for commercial and office buildings. For large-scale projects, this approach enables phased utilization, optimizing resource allocation. Additionally, as construction progresses, more space becomes available around the project, which can be used as car parking spaces and material depots. These advantages are more pronounced in projects with multiple aboveground and underground levels or commercial developments.

2.3.2 Safety

The top-down method is renowned as one of the safest excavation techniques. Unlike traditional methods, construction of the retaining structure precedes excavation in each stage. This proactive approach significantly reduces the risk of excavation-induced deformations. However, it should be noted that the safety of the top-down method relies heavily on meticulous design, construction, and supervision.

2.3.3 Cost Efficiency

A significant cost-saving advantage of the top-down method lies in the elimination of temporary excavation stabilization measures. While there are additional costs associated with this method, the combined costs of excavation stabilization and structural construction are lower compared to traditional methods in most cases. However, it is important to note that cost differences can vary depending on specific project conditions. Generally,

engineers can anticipate cost savings when employing the top-down method.

2.3.4 Time Optimization

The potential time savings of the top-down method depend on various factors, including equipment efficiency, design process optimization, full implementation in both upward and downward directions, effective site management, the number of above-ground and underground levels, and site-specific geotechnical conditions. By optimizing these factors, engineers can significantly reduce construction time.

2.3.5 Legal Compliance

Engineers do not have many options when it comes to stabilizing deep excavations. In some common methods, such as nailing and anchoring, elements of the retaining structure might encroach on adjacent properties. Under Iranian civil law, if one owns a land, one owns the space above and below it as well. This can lead to legal issues if excavation projects encroach on neighboring lands, resulting in delays, disputes, and additional costs. In the top-down method, none of the stabilizing elements is located outside the project site, making it an attractive option for projects with high risk of legal difficulties.

2.3.6 Economic and Environmental Impact

Using the top-down method entails significant benefits for the national economy, addresses Health, Safety, and Environment (HSE) concerns, and improves site management.

By eliminating the need for temporary retaining structures, this method reduces the consumption of construction materials such as steel and cement. This reduction in material consumption can lead to cost savings—particularly when applied to large-scale projects across the country—and environmental benefits as the production of these materials is associated with high greenhouse gas emissions.

Furthermore, the top-down method improves site management by enabling the rapid construction of the first basement floor, providing storage space for equipment and materials, which, in turn, minimizes traffic disruptions and noise pollution. It also provides a shelter for construction workers, improving working conditions and overall project efficiency.

By combining economic benefits, environmental sustainability, and improved project management, the top-down method aligns with principles of sustainable development.

2.4 Weaknesses

While using the top-down method has many benefits, it also presents challenges that engineers should be aware of.

2.4.1 Space Limitations

The confined nature of underground spaces necessitates adjustments to traditional construction procedures. To mitigate the effects of reduced precision in these environments, specialized equipment may be needed, which can be difficult to acquire in many cases. For the same reason, top-down construction often demands more lenient tolerance standards compared to other methods.

2.4.2 Precision Challenges

Specific top-down guidelines determining appropriate tolerance levels are currently lacking, and designers and engineers must use their own expertise and judgment to select the most suitable method for each project. As construction tolerance becomes more lenient in the top-down method, paying extra attention to project interfaces becomes more important. In particular, the interface between structural and architectural designs should be examined carefully, as some structural changes may be necessary during construction. For example, if the distance between columns is insufficient to accommodate parking or ramp requirements, even minor deviations during construction could lead to code violations.

2.4.3 Urban Environment Constraints

Construction using the top-down method can pose significant challenges in urban environments. Underground operations in confined spaces are inherently difficult, and this method may not be viable for projects with limited site dimensions or deep excavation requirements. In such cases, equipment maneuverability is constrained, which can slow down construction operations and lead to potential quality compromises.

2.4.4 Groundwater Risks

Groundwater infiltration is a common problem in underground construction, and the inherent complexities of the top-down method further exacerbate this challenge. Mud formation, reduced equipment maneuverability, difficult soil removal, increased water management requirements, and the need for additional waterproofing and drainage measures are some of the challenges of using the top-down method in water-bearing ground conditions.

2.4.5 Execution Joints

Due to the sequential nature of top-down construction, the formation of numerous execution joints is unavoidable. To mitigate potential issues, strict adherence to building codes for cold joints is essential. This is especially crucial in water-bearing ground conditions, where the risk of water seepage is significantly higher.

2.4.6 Heavy Equipment Restrictions

Efficient application of the top-down method calls for suitable openings for soil removal and equipment access. The location and properties of these openings are determined based on the construction approach adopted by designers and the equipment used for soil removal. Furthermore, the use of steel beams in underground construction can be challenging due to the limited maneuverability of cranes in confined spaces, which makes the transportation and installation of these beams and other heavy elements difficult. As such, even in concrete structures, slab-column systems are preferred to facilitate ease of construction.

Given these limitations, the feasibility of the top-down method should be assessed on a case-by-case basis for each individual structure.

2.5 Feasibility Criteria

To ensure the successful application of the top-down construction method, a thorough assessment of feasibility is essential. Key criteria—ranging from architectural and structural considerations to geotechnical and financial factors—must be evaluated to determine if this method aligns with the unique demands of each project.

2.5.1 Architectural Considerations

The more lenient tolerance requirements necessary in the top-down method can lead to greater deviations between the as-built conditions and the original design plans. While some deviations are inevitable regardless of the adopted method of construction, top-down deviations are both more common and more significant. The impact of potential deviations on the function and quality of a structure should be evaluated before construction begins. Architectural considerations—such as compliance with municipal regulations, parking space requirements, underground passage dimensions, minimum ramp widths, and other critical measurements—must be carefully evaluated. Potential deviations from these standards and their impact on the structure's function should be thoroughly assessed.

2.5.2 Structural Considerations

Providing enough lateral stiffness through diaphragm wall design between basement floors is a critical consideration engineers need to keep in mind when using the top-down method. Given the fact that floor diaphragms act as struts, making sure they have enough stiffness to control deformations is of utmost importance. The in-plane stiffness of slabs varies depending on the ceiling type, with concrete slab floors—such as flat and waffle slabs—exhibiting greater stiffness, while EPS joist slabs offer less. In most cases, the stiffness remains sufficient; however, it may be compromised in

areas with openings. Therefore, the size and placement of these openings must be carefully considered. In addition to skylights, staircases, elevators, and facility shafts, diaphragm openings include areas where rigidity is reduced, or structural elements are located outside of slabs, e.g. ramps. In these instances, rigidity fluctuations must be evaluated carefully. Slab openings for soil removal and equipment access follow the same principle. In smaller projects with limited floor areas and small diaphragms, multiple openings can cover significant portions of slabs, greatly reducing stiffness. Even facility openings—especially those of the strip type—can reduce stiffness considerably despite their small size.

2.5.3 Geotechnical Considerations

Unlike the conventional approach, the SPW approach to top-down construction does not incorporate integrated diaphragm walls. Instead, soil's stand-up capacity in spaces between lateral columns is used in each excavation stage before a retaining wall is built. Therefore, the SPW approach to the top-down method is implausible in soft soil conditions. In such conditions, using a diaphragm wall or constructing other types of continuous embedded retaining walls according to the conventional top-down method is inevitable. Geotechnical engineers assess the soil's suitability for the SPW method based on its stand-up capacity.

2.5.4 Other Construction Considerations

A critical factor in determining the feasibility of the top-down method is the availability of suitable openings for soil excavation and removal. This assessment must consider the surrounding buildings and infrastructure to ensure that the equipment can access the site and remove soil efficiently. For example, if a lattice-boom crawler crane is selected for soil removal, it must be possible to install and operate the crane through the available openings.

2.5.5 Financial Considerations

While financial considerations are not, strictly speaking, among feasibility criteria, they should be taken into account when selecting the appropriate construction method. There are no rules

applicable to all situations, and top-down method's financial feasibility compared to other methods should be evaluated on a case-by-case basis, following a rigorous analysis of different design and construction strategies.

2.6 Classifying Structures Based on Top-Down Construction Feasibility

Steel, concrete, and composite structures can all be built using the top-down method, though each has its own characteristics and limitations.

Constructing embedded steel columns using the top-down method is straightforward. In concrete structures, columns are designed and constructed as SRCs, where the steel core is embedded within the shaft, followed by the sequential construction of the reinforced concrete casing. Precast concrete columns can also be installed using the top-down method, although their weight can make their transfer and installation difficult. For deep excavations with long columns, these columns should be constructed stage-by-stage and spliced at shaft heads. Splicing precast concrete columns at shaft heads can be difficult due to their weight compared to steel columns. Cast-in-place concrete columns can be installed within shafts using formwork and concreting procedures, but this method is typically limited to shallow excavations and requires specific design and construction considerations.

The use of steel beams in top-down construction can be challenging due to limitations in crane access and maneuverability in basement levels. Even in concrete structures, slab-columns systems are preferred. Among different ceiling systems in steel or concrete structures, those without a subbase—such as ram and scaffold systmes—make for faster construction times. However, it is crucial to select a ceiling system that can provide adequate structural stiffness.

2.7 Top-down Method's Applicability

The top-down method is a versatile construction technique applicable to various structures. While it is commonly associated with high-rise buildings, it has also found significant application in non-building sectors. Metro and train stations are ideal candidates for the top-down method due to factors such as limited space, the need to minimize traffic disruptions, large structural dimensions, and the requirement for precise deformation control. Additionally, tunnels and grade separation projects benefit from the top-down method's ability to manage traffic and minimize disruptions during construction.

This is sample content for Chapter 2.

Chapter 3

Key Concepts of Top-down Construction

Chapter 3: Key Concepts of Top-down Construction

3.1 Introduction

As demonstrated in the previous chapters, the top-down method as an approach to construction involves multiple execution stages that vary based on specific characteristics of each project, including financial factors, available equipment, geotechnical conditions, and structural and architectural plans. The core principle of the top-down method is the construction of vertical load-bearing components as embedded elements in the soil, following which a floor slab is constructed as a lateral bracer against soil-induced stress to make excavation and underground construction possible. Different stages of the top-down method and construction considerations specific to each stage will be described in this chapter.

3.2 Construction Stages

In general, top-down construction comprises the following stages:

1. Excavation to prepare the site for the installation of the vertical load-bearing components
2. Building structural columns and embedded retaining walls.
3. Excavating the underground space.
4. Constructing underground structures.
5. Constructing aboveground structures.
6. Underground, top-down construction of various structural components.

The subsequent sections of this chapter will delve deeper into each stage, exploring construction nuances and best practices.

3.2.1 Preparing the Site for the Construction of Vertical Load-bearing Components

In the first stage, excavation operations are carried out to prepare the site for the installation of vertical load-bearing components like columns and embedded retaining walls. Depending on the construction method, various approaches to excavation can be adopted. It can be carried out manually by construction workers, or heavy machinery can be used to establish shafts and trenches. The choice of equipment depends on the type and design of load-bearing components, geotechnical conditions, scale of the project, available equipment, and financial considerations.

3.2.1.1 Manual Excavation

Manual excavation refers to using construction workers and basic equipment such as pickaxe, mallet, pneumatic or electric DTH (Down-The-Hole) drills, and manual or electric winches to excavate shafts, galleries, or trenches. This is one of the most elementary methods of excavation, commonly employed in regions with low labor costs. Despite its simplicity, manual excavation can be adapted to various excavation scenarios, as discussed below.

3.2.1.1.1 Manual Excavation Applications

3.2.1.1.1.1 Shaft Excavation

Shaft excavations are undertaken to prepare the site for the construction of columns and retaining walls. To prevent cave-ins and protect workers in challenging geotechnical conditions, it is advisable to implement tubing and other stabilization measures as excavation operations are ongoing. Tubing involves the use of precast reinforced concrete elements in elliptical, circular, or squared shapes to stabilize shaft walls. Alternatively, in-place reinforced mesh or shotcrete can be used for stabilization. While manual shaft excavation is readily available in most cases, it can be time-consuming and pose significant safety risks to workers.

The cross-sectional shape and location of shafts relative to structural columns are critical design considerations. The cross-sectional shape can be square, circular, or any other suitable configuration. The optimal cross-sectional shape for shafts can vary depending on the location of the column, whether it is a middle or peripheral column. For middle columns, the shaft can be aligned with the column axis, allowing for sufficient clearance between the column and shaft walls. Here, the column is positioned within the shaft, without direct contact with shaft walls. Consequently, the specific shape of the shaft's cross-section is less critical, as long as it provides sufficient space for the column and its connections, and allows for necessary installation tolerances, particularly when using labor-intensive construction methods.

Peripheral shafts, on the other hand, require closer integration between the column and shaft wall to ensure effective soil retention. In these cases, close contact between the column and shaft wall is essential to ensure effective soil-structure interaction. Therefore, peripheral shafts cannot be aligned with the column axis, as the columns must be positioned on the outer edge of the shaft to provide lateral support to excavation walls (Figure 1.3). The excavation walls may need to be modified from circular to a more rectangular or horseshoe shape to accommodate the column placement and provide adequate support. But square cross-sections are not common for shafts and suitable precast tubings are not always available. All things considered, the form and size of the shaft's cross-section as well as its placement in the design plan relative to columns are execution considerations and all execution factors, including the column's placement in the plan, column's distance to excavation walls, column's placement relative to piles, pile cage access, tube access for pile concrete casting, and worker access must be taken into account based on construction needs and column installation method in the shaft. For peripheral columns, the physical contact between the column and the soil wall is crucial for the successful implementation of the top-down method.

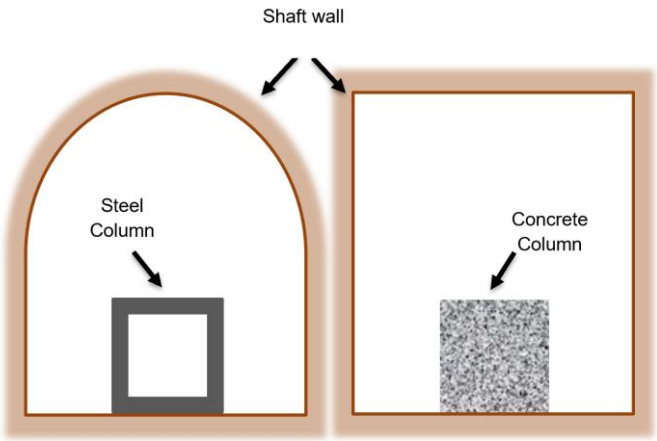


Figure 3.1 Shaft and column placement

Safety is paramount during manual shaft excavation. Encountering underground water sources, such as wells, qanats, or sewage systems, can increase the risk of cave-ins and endanger workers. We will address specific safety considerations in Chapter 5. Additionally, proper sealing of shaft heads is crucial in top-down construction where multiple shafts are typically present at a project site.

Manual shaft excavation is only feasible in soil conditions with sufficient stability. High groundwater levels or the presence of water streams can render manual excavation impractical. Moreover, manual excavation below the water table or in areas with significant water seepage is challenging and not recommended.

The construction of large diaphragm wall panels—usually, square-shaped—in the conventional top-down method typically involves mechanized installation techniques using bentonite-stabilized liquids. Manual excavation should only be considered under the direct supervision of a geotechnical engineer and is generally not recommended for large-scale trenches due to the associated risks.

3.2.1.1.1.2 Gallery Excavation

Galleries are horizontal passages excavated in the soil for various purposes. Manual excavation of galleries can be hazardous and should only be undertaken under expert supervision, and gallery excavation in unstable soil conditions should be avoided. All safety considerations for shaft excavations also should be used to galleries. To enhance stability and safety during excavation, egg-shaped or elliptical tubing elements should be installed along the entire length of the gallery. Grouting the space between the gallery and the surrounding soil can further improve stability. The gallery ceiling should be excavated in arched sections, with a maximum height of 1.50 meters and a maximum width of 1.20 meters. In the SPW method, galleries are often constructed to improve worker safety and facilitate access between shafts. When galleries are used as shelters for workers during column installation, they should be designed to accommodate workers comfortably and stabilized with appropriate support systems, such as tubing.

3.2.1.1.1 Mechanized Excavation

Mechanized excavation methods vary depending on the specific top-down construction approach employed. For instance, in the conventional approach, trench excavation is necessary to prepare the site for diaphragm wall construction, while the SPW approach typically does not involve trench excavation. The selection of excavation machinery is influenced by factors such as site dimensions and the feasibility of transporting, installing, and operating the equipment. In projects with limited space, larger machinery may not be suitable. The following sections will discuss various types of excavation machinery.

3.2.1.2.1 Excavation Machinery

3.2.1.2.1.1 Pile Boring Machines

Rotary excavation machines use a drill bit to create a circular shaft (Figure 3.2). Soil is removed through an opening in the machine's arm from the shaft when the direction of drill rotation shifts. The diameter of the drill bit determines the shaft's diameter, and the machine's power and type dictate the maximum achievable width

and depth, which can be up to 4 meters and 120 meters, respectively.

In challenging ground conditions, such as soft, unstable soils or high groundwater levels, steel casings can be embedded in the ground to stabilize the shaft walls and enable excavation.

A key difference between mechanized and manual shaft excavation is the degree of disturbance to the shaft walls. Mechanized excavation, particularly using rotary drilling methods, can result in significant wall instability, including indentations, protrusions, and localized failures. Unlike manual excavation, where wall stabilization can be performed at the same time, mechanized excavation often requires additional stabilization measures, such as casing installation, to prevent wall collapse. Until the shaft walls are adequately stabilized, workers should avoid entering the shaft. Other safety precautions outlined for manual excavation should also be strictly followed.



Figure 3.2 A rotary pile boring machine.

3.2.1.2.1.2 Hydraulic or Mechanical Grabs

To excavate a rectangular trench for diaphragm wall construction, hydraulic or mechanical grabs can be employed. These grab attachments, which can be mounted on cranes or other hydraulic machinery (Figure 3.3), use two mandibles to dig and remove soil. The maximum excavation depth is typically limited to around 70 meters, depending on the crane's capacity. The width of the excavated trench can range from 60 to 150 centimeters, and its length can vary from 240 to 420 centimeters, depending on the specific grab module.



Figure 3.3 A grab module attached to a crane.

Depending on the trench dimensions and ground conditions, bentonite slurry may be used to stabilize the excavation. As excavation progresses, bentonite slurry is continuously circulated to prevent soil collapse. However, in cases where the ground conditions are favorable, such as the presence of rigid soil layers, the use of bentonite slurry may be minimized or avoided altogether after consulting experts, potentially reducing project duration.

3.2.1.2.1.3 Hydromill or Hydrofraise

Hydromill or hydrofraise machines are another type of equipment capable of excavating rectangular trenches. These machines, which can be mounted on a crane, utilize two vertical rotating wheels to excavate the soil, and at the same time, use a suction and vacuum mechanism to remove the excavated material (Figure 3.4). Hydromills can be used for excavations up to 250 meters deep. The width and length of the excavated panels, which typically range from 60 to 200 centimeters and 260 to 320 centimeters, respectively, are determined by the specific hydromill module. Similar to hydraulic or mechanical grabs, hydromill excavation often involves the use of a stabilizing fluid to prevent soil collapse during the excavation process.



Figure 3.4 Hydromill module attached to a crane.

3.2.2 Construction of Vertical Load-bearing Components

In the top-down method, all vertical load-bearing elements, including columns and retaining walls, are supported by an initial foundation system. This initial foundation, typically composed of single piles, transfers dead loads, soil pressures, and live loads to the underlying ground. The design of these piles must adhere to

relevant building codes to ensure adequate load-bearing capacity and settlement performance.

Following shaft excavation, the construction of the initial foundation is the next critical step. While other foundation types, such as isolated footings, are possible, piles are often preferred due to their ability to restrict differential settlement, their independent performance, and the challenges associated with quality control in confined shaft environments. The design and performance of the initial foundation are paramount, as the structure is especially susceptible to settlement during construction.

3.2.2.1 Construction of the Initial Foundation

Given that piles are commonly used for initial foundation systems, the subsequent discussion will focus on pile construction within the context of top-down construction. To ensure a seamless transition to the final foundation, specific guidelines must be followed during construction.

3.2.2.1.1 Pile Reinforcement Cage Configuration and Installation

Based on the top-down construction design, pile reinforcement cages can be square, rectangular, or circular. To maintain their structural integrity during handling and installation, reinforcement cages may require additional support elements, such as chair bars or steel frames.

In top-down construction, columns can be attached to the initial foundation in various ways. The most common and recommended method involves driving structural columns into pile heads and anchoring them in place (Figure 3.5). This approach is suitable for steel, precast concrete, and the steel cores of composite concrete columns.

Alternatively, base plates with anchor bolts can be placed on pile heads, and columns can be attached using base plates and additional anchor bolts (Figure 3.6). However, this method presents safety concerns due to potential quality control issues

resulting from limited access to column shafts while they are suspended from the crane.

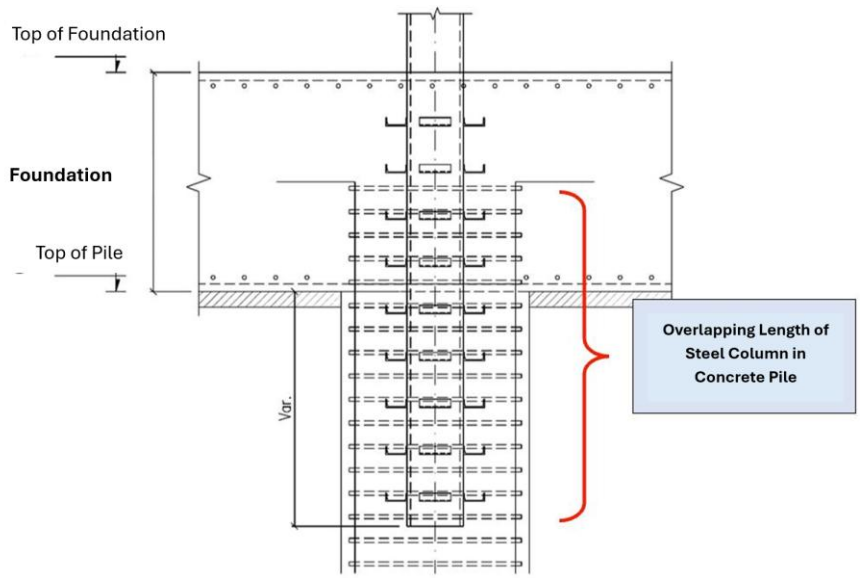


Figure 3.5 Detail of the connection between a steel column or a composite column's steel core to a pile; embedded connection

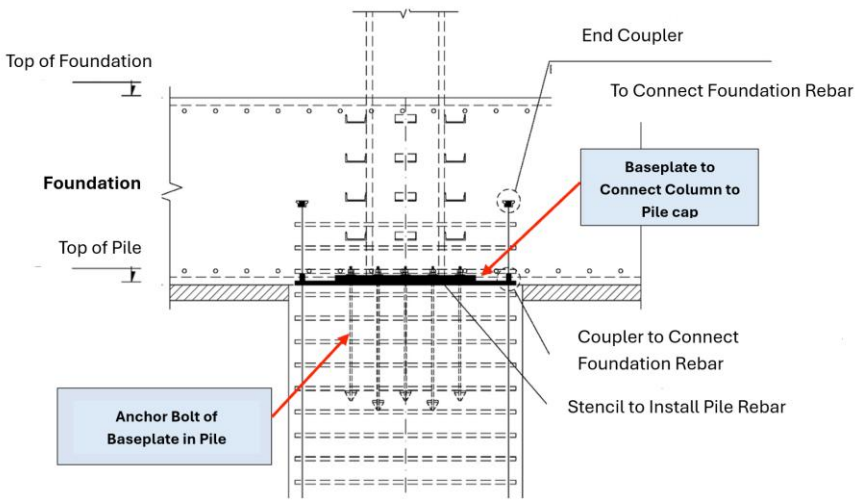


Figure 3.6 Detail of the connection between a steel column or a composite column's steel core to a pile, base plate to pile head connection

The configuration of the reinforcement cage within the pile can vary depending on the specific column-to-pile connection method. For columns connected to piles using embedded anchor bolts, the pile cage and column base plate anchor bolts must be positioned within the shaft before concrete casting.

Figure 3.7 illustrates a circular pile cage configuration prepared for a connection similar to that shown in Figure 3.5. In this scenario, the pile cage is first lowered into the shaft, followed by the plunge installation of the column on the shaft head. After concrete casting, the anchoring length of the column is integrated with the pile.



Figure 3.7 Pile cage in a circular connection used to install a plunge, anchored column to a pile head, transverse reinforcement consists of spirals or closed stirrups. For more details, refer to Figure 3.5

Figure 3.8 depicts a rectangular pile cage configuration, where steel frames positioned on the two ends of the cage are used to maintain its shape and stabilize the column base plate anchor bolts. After concrete casting, these steel frames are removed and reused in the fabrication of additional base plates. This approach

helps to minimize the displacement of anchor bolts and ensure accurate alignment between the column base plate and the pile head, facilitating a proper column connection.
The design assumptions for the pile-foundation connection will influence the required reinforcement within the pile. For a fixed connection, the pile reinforcement should be extended into the foundation to ensure adequate anchorage.



Figure 3.8 Pile cage in a square configuration used with steel frames to improve rigidity and integrity; anchor bolts are used to connect the base plate to the pile reinforcement cage

Typically, pile rebar cages have a circular configuration. To accommodate plunge, anchored columns, transverse reinforcement bars, often in spiral shapes, are incorporated into the pile design, as illustrated in Figures 3.5 to 3.9.

As shown in Figure 3.10, the pile reinforcement is typically anchored into the foundation to a depth of half the foundation thickness, which should be accounted for in both foundation design and cage configuration.



Figure 3.9 Pile cage in a circular configuration used to install a plunge, anchored column to a pile head; spiral transverse reinforcement. For more details, refer to Figure 3.5

To ensure worker safety during rebar cage and column installation, plastic safety caps can be placed on the ends of pile reinforcement bars.

Figure 3.10 demonstrates how steel frames are attached to pile cages.



Figure 3.10 Steel frame in a base plate with anchor bolts used for connection to pile rebar cage

Mechanical rebar splices or couplers are used to extend and connect pile reinforcement bars. Once the excavation reaches the foundation level, the pile reinforcement is anchored to the foundation using mechanical splices.

In some cases, a fixed connection between the pile and foundation may not be required, making the extension of the pile reinforcement into the foundation unnecessary. However, the design, modeling, and analysis of the foundation and piles must account for this connection type, ensuring that the pile-to-foundation connection adheres to the specified design assumptions. Any modifications to the pile-foundation connection should be carefully reviewed and approved by the designer to ensure the integrity of the top-down structure.

Subject to the designer's approval, the transverse reinforcement in circular piles can be configured as closed or single-legged stirrups, or as spirals. For spiral reinforcement, the spacing of the spirals should be determined based on applicable building codes and the designer's specifications, considering the required concrete casting space. Without the designer's consent, it is not permissible to change the specified transverse reinforcement type from closed stirrups to spiral reinforcement.
The design of pile cages and the placement of columns should consider the required access for concrete casting. This may involve installing a tube within the pile cage with a minimum diameter of 5 inches. The pile design and reinforcement configuration—whether it is a closed stirrup or single-legged stirrup—must accommodate this tube (Figure 3.11).

To ensure adequate concrete cover for pile walls and floors, appropriate plastic or concrete spacers should be used. For shaft walls and their surrounding areas, specialized spacers known as "cage fixes" are recommended (Figure 3.12). Cage fixes are wider and more stable than traditional wheel spacers, and their flat surface facilitates the smooth passage of the pile cage through the shaft.

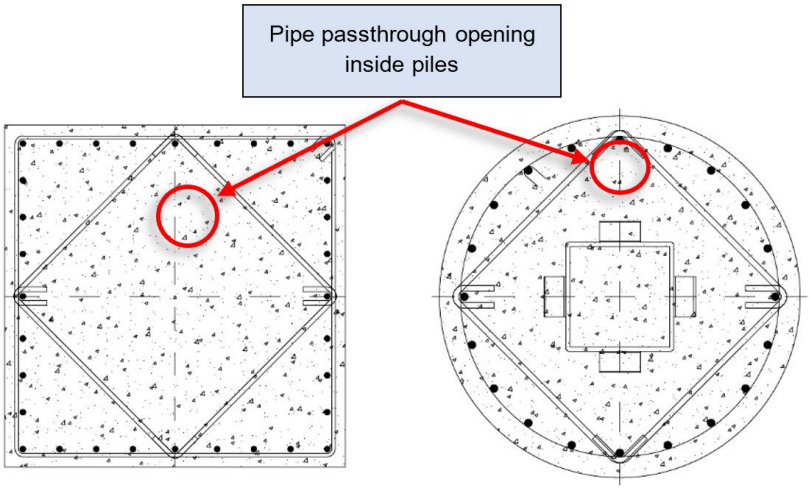


Figure 3.11 Cross-section of a pile showing the arrangement of transverse reinforcement bars relative to the pile and the embedded column, and the provision of a concrete casting access hole.



Figure 3.12 A cage fix used to ensure adequate concrete cover for pile.

3.2.2.1.2 Pile Concrete Pouring

Due to the distance between the concrete pouring location and the excavation edge, concrete should be poured using the tremie method to minimize the free-fall height and ensure compliance with construction standards.

Concrete should be poured continuously to avoid cold joints. Multi-stage concrete placement is not permitted in piles. The tremie tube used for concrete placement must comply with relevant national and international standards. Refer to Figures 3.13 and 3.14 for more details.

If groundwater is present in the shaft during concrete placement, it should be removed using pumps designed for the specific water flow rate in the shaft. If water removal is not practical, the tremie method can still be employed. In this case, however, an initial concrete placement using the tremie tube is crucial to displace the water and ensure proper concrete placement. While both dry and wet tremie methods are described in various guidelines, the wet tremie method is generally preferred due to its effectiveness and ease of implementation.

During the initial stage of dry concrete casting, a steel plate with a sealing ring is placed at the bottom of the tremie pipe to prevent water ingress as the pipe is lowered into the shaft. At this point, the tremie pipe is sealed, and the concrete only comes into contact with water when the casting process begins. In essence, the concrete flows through a dry pipe. To facilitate concrete flow, the tremie pipe is gradually raised by 10 to 20 centimeters, allowing the concrete to exit from the bottom of the pipe and settle on the shaft floor. In shafts with high water levels, hydrostatic pressure can exert an upward force on the tremie pipe, potentially causing buoyancy.

During the initial stage of wet concrete casting, a plug or separator is placed inside the tremie pipe to prevent the mixing of water and concrete, which can lead to concrete segregation. To initiate the

casting process, the tremie pipe is lowered to the bottom of the shaft and then raised slightly to create a small gap between the pipe's end and the shaft floor. The height should not exceed the pipe's diameter. The plug is then removed to allow the concrete to flow into the shaft.

During underwater concrete casting, the tremie pipe should be submerged at least 3 meters into the concrete. If the pipe is submerged more than 3 meters, the casting process can become challenging. Therefore, the pipe should be maintained at a depth of 3 to 8 meters. It is recommended to raise the pipe by 1 meter after casting 3 to 4 meters of concrete—the length of the pile shaft—to complete the remaining portion. At the end of the casting process, the pipe should be gradually withdrawn to a depth of 2 meters.

Ideally, concrete should flow smoothly through the tremie pipe during the casting process. Excessive shaking, raising, or lowering of the pipe can disrupt the concrete flow and increase the risk of mixing clean and dirty concrete, particularly in underwater casting operations. To ensure smooth concrete flow, the concrete pressure must overcome pipe friction, hydrostatic pressure, and the resistance from the previously placed concrete.

Since vibration cannot be used to remove air bubbles from concrete during pile casting, the concrete mix design should prioritize self-compacting. The mix should be designed to achieve a balance of stability, viscosity, and yield stress to minimize segregation and bleeding. To further reduce segregation and bleeding, the following measures are recommended:

- Replacing a portion of cement with admixtures of finer particle sizes
- Optimizing aggregate gradation
- Adding plasticizers or superplasticizers
- Adjusting water or paste content

Furthermore, appropriate admixtures can be used, provided they comply with relevant building codes and standards.

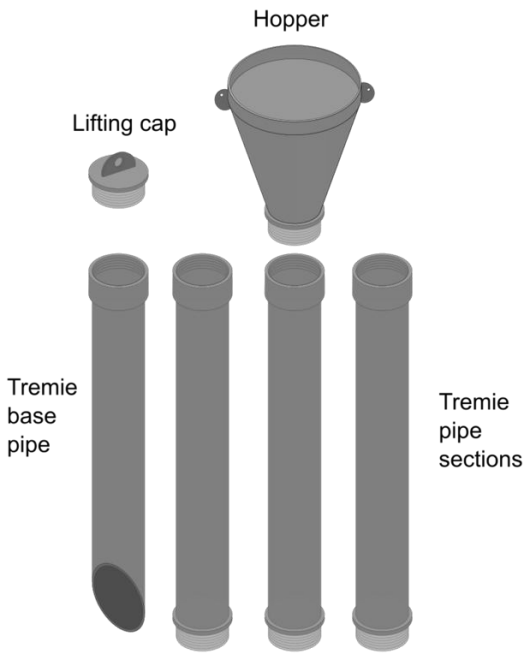


Figure 3.13 Components of a tremie pipe

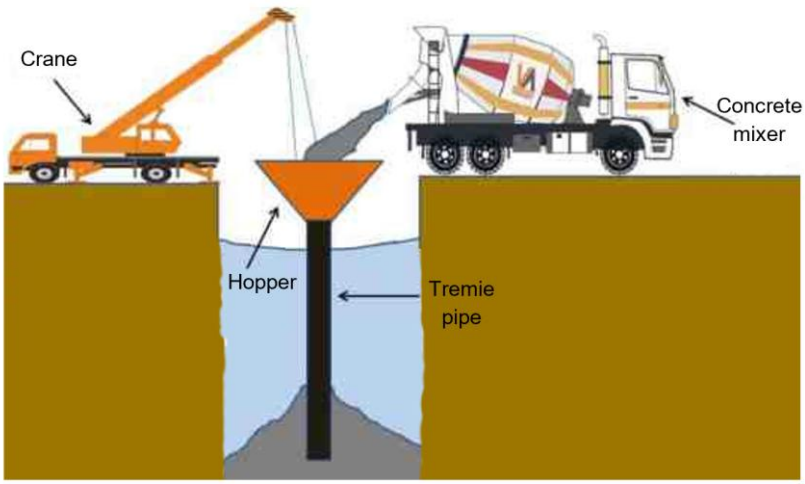


Figure 3.14 A schematic view of underwater concrete casting using the tremie method

Figure 3.15 illustrates the installation of a plunge column on a pile head. A 5-inch diameter hose can be connected to a ground line pump or a boom pump and lowered through the gap between the column, shaft wall, and pile reinforcement to facilitate concrete placement. Once the concrete is placed, the hose can be removed using a crane.

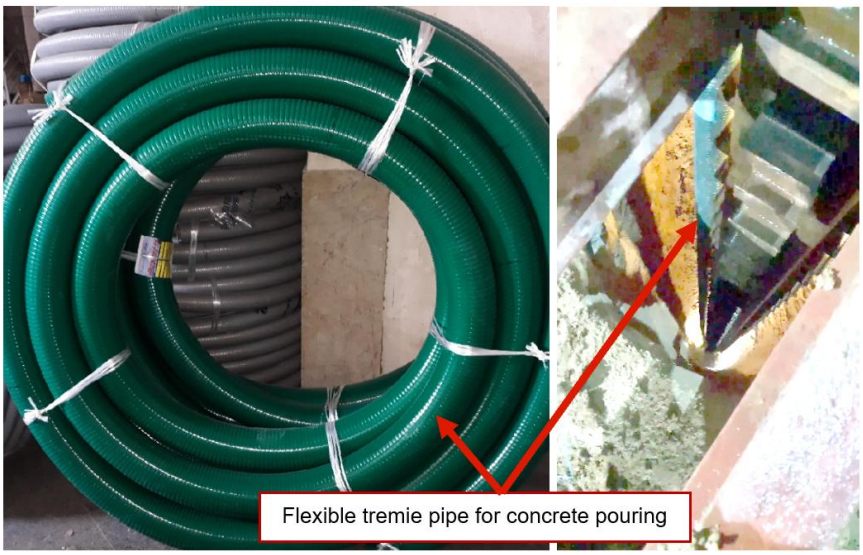


Figure 3.15 A 5-inch diameter concrete pouring hose lowered to the bottom of a pile shaft

3.2.2.1.3 Preparations for Installing Vertical Load-bearing Elements

Before installing vertical load-bearing elements in a top-down construction project, several preparatory steps are necessary. These preparations may be required at the shaft head, within the shaft, or at the pile head, and vary depending on the specific installation method. The following sections will outline the preparatory steps for different types of vertical load-bearing elements.

3.2.2.2 Installing Columns

3.2.2.2.1 Shaft Head Preparations for Installing Plunge Columns

Installing and plumbing columns in a top-down construction project can be a complex procedure that requires careful attention

to detail. To minimize errors and ensure safety, it is advisable to use appropriate shaft head elements. The size and capacity of these elements will depend on the specific construction method as well as column weight and shape.

For plunge-connected columns embedded in piles, the shaft head element must be designed to support the weight of the column during installation and transfer the load to the underlying soil, allowing for precise alignment and plumbing. The complexity of these elements can vary depending on the specific design and construction requirements. Some designs may involve simple supports, while others may require more sophisticated solutions.

Preparing the shaft for plunge column installation typically involves the following steps:

1. Removing excess soil and leveling the shaft head soil manually
2. Constructing a foundation with adequate load-bearing and settlement properties
3. Installing the appropriate shaft head elements on the foundation
4. Installing the plunge column and securing the connections

The stability of the shaft wall, shaft head, and all associated elements, including connections, must be carefully analyzed and designed to withstand the loads imposed by the plunge column.

Figure 3.16 shows a basic shaft head element used to support a plunge column during installation. Concrete pads are constructed on opposing sides of the shaft at the desired level. Steel beams span the pads, supporting the column's seating connections (welded in place). To maintain column plumb during concrete pouring, an anchoring system is installed above the pile head within the shaft, based on specific construction conditions and the engineer's recommendations.



Figure 3.16 Plunge column installation on a shaft head

Figure 3.17 illustrates a plunge column frame used to install precast concrete columns. Given the significant weight of precast concrete columns, stabilizing them during installation can be challenging. Preventing internal deformations under heavy load from the column, the robust steel frame shown in Figure 3.17 provides the necessary support and stability. Once placed in its designated point, the frame is connected to the column using high-capacity steel cables. The frame's components facilitate column alignment and secure the column during installation. By adjusting the height of the screws, the column can be precisely positioned and plumbed. This frame can also be used for the installation of steel plunge columns.



Figure 3.17 Shaft head frame used to install precast concrete plunge columns and steel plunge columns

Figure 3.18 demonstrates a variation of the plunge column frame used in countries other than Iran, which extends both above and below the shaft head. When the column is lowered into the shaft, the frame attached to its top is placed on the shaft head, enabling precise column alignment and plumbing. A more sophisticated variation of this frame is demonstrated in Figure 3.19, incorporating precise leveling mechanisms and hydraulic rams at different depths to achieve greater accuracy.

When using shaft head frames, it is important to ensure that the tremie pipe can pass through the frame to allow for pile concrete casting. Once the concrete has set and the column head is secured, the shaft head frame can be removed.



Figure 3.18 A non-local variation of shaft head frame used to install plunge columns

As of this writing, advanced shaft head installation equipment, similar to those shown in Figures 3.18 and 3.19, is not readily available in Iran. Given the significant height of many columns and the potential for movement during pile concrete casting, it is crucial to anchor the column within the shaft, particularly near the pile head. This is especially important for steel columns, which are more susceptible to movement due to their lower weight compared to precast concrete columns. By anchoring the column to the shaft wall, stability can be maintained and precise alignment can be ensured.

Figure 3.20 illustrates a plunge column within a shaft, where the column is anchored to the shaft wall using a rectangular hollow profile and steel sheets, approximately 3 meters above the pile head, to enhance stability and plumbing during the installation process.

Preparing the shaft head for base plate column installations is a relatively straightforward process. No special preparations are

typically required, except in cases where deep excavations necessitate splicing the column into sections. In such instances, plunge column frames, similar to those described earlier, can be used to facilitate the splicing process.

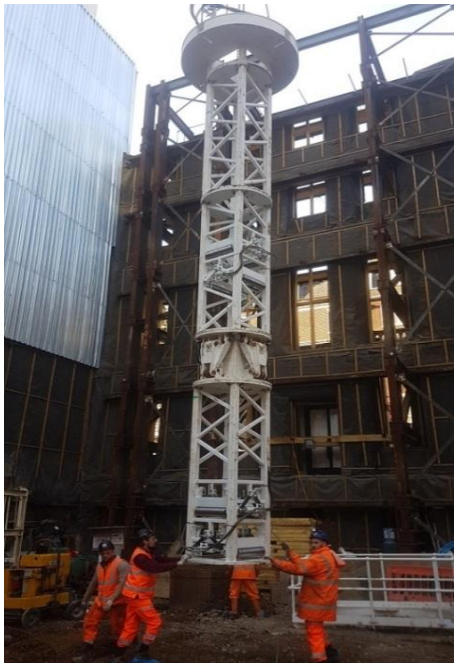


Figure 3.19 A non-local variation of the shaft head frame used to install plunge columns, incorporating hydraulic rams and a sophisticated stabilizer and leveler system

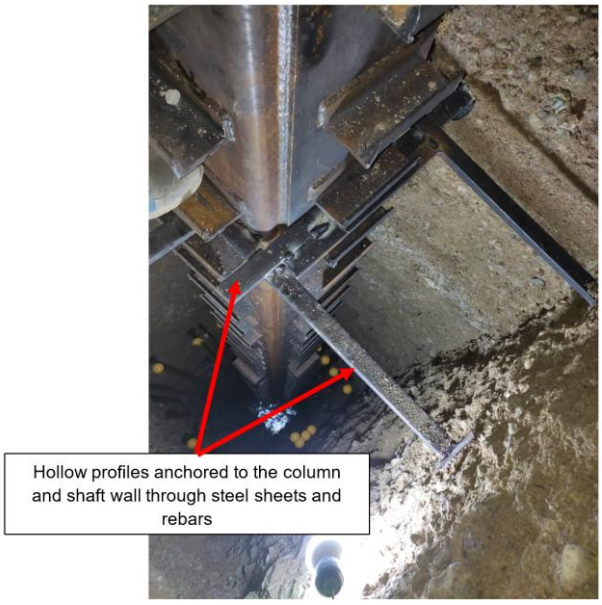


Figure 3.20 Anchoring a plunge column to the shaft wall using two orthogonal rectangular hollow profiles welded to the column

Figure 3.21 illustrates the welding process for splicing a column in a top-down construction project. The bottom part of the column is first installed and stabilized using a plunge column frame. Then, the top part is lowered into place and welded to the bottom section.

Figure 3.22 demonstrates a bolted splicing method for columns in top-down construction. Bolted splicing offers a faster alternative to welding.

Figure 3.23 shows shaft head splicing for precast concrete columns. The bottom part is installed using a plunge column frame, while the top part, suspended from a crane hook, is lowered into place and bolted to the bottom part using flanges.

Figure 3.24 illustrates an alternative method for splicing precast concrete columns where flanges and bolts are replaced by couplers and grout. Reinforcement bars of the two parts are spliced together using couplers, and a steel frame is installed around the joint to facilitate grouting and create a monolithic concrete section.

Splicing design must consider practical construction constraints. It is strongly recommended to perform splicing operations at the shaft head, particularly for welded connections. Welded connections require specialized expertise and equipment, and performing such work within the confined space of a shaft can be challenging and may compromise quality. Additionally, the limited space within a shaft can hinder precise welding operations.

Columns longer than 12 meters may require specialized transport trucks. However, with proper traffic and safety measures, columns up to 24 meters can be delivered to the site, reducing the need for on-site splicing or minimizing the number of required splices.

Splicing columns at the manufacturing facility often produces higher-quality connections than on-site splicing at the shaft head while also expediting installation, regardless of whether the

plunge method or base plate installation is used. However, transporting longer columns can be costly, requiring a thorough technical and financial analysis to determine whether the time savings justify the additional expense. Project-specific factors such as passage widths, transport logistics, and site conditions play a crucial role in assessing the practicality of installing long, single-piece columns. In some cases, their installation within the shaft may not be viable.

To summarize, various approaches can be considered for column installation, including:

- **Factory Splicing:** Splicing columns at the manufacturing facility, followed by transportation to the site. This method generally results in higher-quality connections.
- **On-Site Workshop Splicing:** Splicing columns in an on-site workshop, followed by installation in the shaft. This approach offers better quality control compared to in-shaft splicing.
- **In-Shaft Splicing:** Splicing column sections within the shaft. This method is more challenging and may compromise quality.

Factory splicing typically results in the highest quality connections. However, it may involve higher transportation costs and require specialized equipment for handling and transporting the longer columns. On-site workshop splicing offers a balance between quality and practicality, but may still require additional equipment and careful planning. In-shaft splicing is generally the least desirable option due to the challenges associated with working in confined spaces and the potential for compromised quality.



Figure 3.21 An example of welded splicing of a steel column at the shaft head

When the base plate and column are installed simultaneously, the anchor bolts on the base plate must be secured at the shaft head and below the footing. In such cases, as long as there is no need for splicing, the focus shifts from shaft head preparations to pile head preparations. As illustrated in Figure 3.10, steel frames can be used to stabilize the anchor bolts within the pile cage. Once pile concrete is cast, these frames can be removed and used for fabricating base plates.

In all these scenarios, a simple support system can be used to stabilize the column at the shaft head and keep it plumbed as the column's weight is directly transferred to the pile. This system typically involves a pair of rectangular profiles and grout. Figure 3.25 demonstrates an example of this approach in a construction project. However, the specific design and implementation of the shaft head support system should be based on the project's design requirements and the engineer's recommendations.



Figure 3.22 Bolted splice connections at a shaft head on a plunge steel column



Figure 3.23 Splicing on a precast concrete column using steel flanges and bolts



Figure 3.24 Splicing on a precast concrete column using couplers and grouting



Figure 3.25 Pile anchors after concrete casting; steel frames have been removed

3.2.2.2.2 Construction and Installation of Columns

The top-down method is compatible with a variety of column types, including steel columns (boxed, tubular, H, and cross configurations), precast concrete columns, cast-in-place concrete columns, and composite columns including concrete-filled tubes (CFT) and steel-reinforced concrete (SRC) columns.

3.2.2.2.2.1 Construction and Installation of Steel Vertical Load-bearing Components

The selection of steel columns should consider financial factors, structural design requirements, and construction constraints. Steel columns can be used as working load-bearing elements in top-down construction, provided they are designed to withstand specific loading conditions and construction procedures. After preparing the shaft head and installing the pile cage, steel columns can be installed and anchored to the pile.

Steel sections can also be used as cores for composite concrete columns. The construction and installation of these composite columns are similar to those of steel columns. However, the steel core's primary function is to transfer loads during construction, while the final composite section (steel core and concrete) is designed to carry the ultimate working loads.

The first step after structural design is to develop detailed shop drawings. These drawings should include all installation requirements for columns and consider manufacturing constraints. Whether components require prefabrication or on-site assembly, the structural design and shop drawings must be coordinated accordingly.

A common approach to top-down construction involves embedding a steel column within a concrete pile, as illustrated in Figure 3.5. In this method, the column is first plunged into the shaft, followed by pile concrete casting. Shear connectors, such as head studs and C-channels, are used to transfer loads between the

steel column and the concrete pile. This approach is relatively low-risk and straightforward, but careful design considerations are necessary to ensure adequate load transfer. It's important to note that head studs are generally preferred over C-channels as shear connectors because they can effectively resist forces from multiple directions. For C-channels, installation must be carried out with the direction of shear flow in mind.

For steel columns installed with base plates, the initial alignment is established at the shaft head. Using a plumb bob and manual methods, reference points are transferred from the shaft head into the shaft and to the pile head. The base plate is then positioned on the bolts, and the column is aligned and secured using light elements. Figures 3.26 and 3.27 illustrate this process. To minimize risks of gas poisoning and improve quality, it is generally recommended to avoid any welding activity within the confined space of the shaft. Instead, the column can be pre-assembled with the base plate and lowered into the shaft as a single unit. Figure 3.28 shows an overhead view of a column installed using this method.



Figure 3.26 Transferring alignment reference points from the shaft head to the pile head to install a column bolted to a base plate



Figure 3.27 Installation and grouting of a column and its base plate on bolts anchored to a pile

If additional components are required to be prefabricated for column connection, these should be incorporated into the top-down design and shop drawings. Figure 3.29 shows a column with pre-welded seating angles at the design installation level, which facilitates the plunge installation process. These seating angles were designed to withstand the loads associated with column installation. Figure 3.30 illustrates another example where four pre-fabricated steel brackets as well as guide slides are installed on a column at the splicing point to aid in the initial alignment and connection of two column sections before welding.



Figure 3.28 Steel column installation on base plate bolts at pile head in a top-down construction project

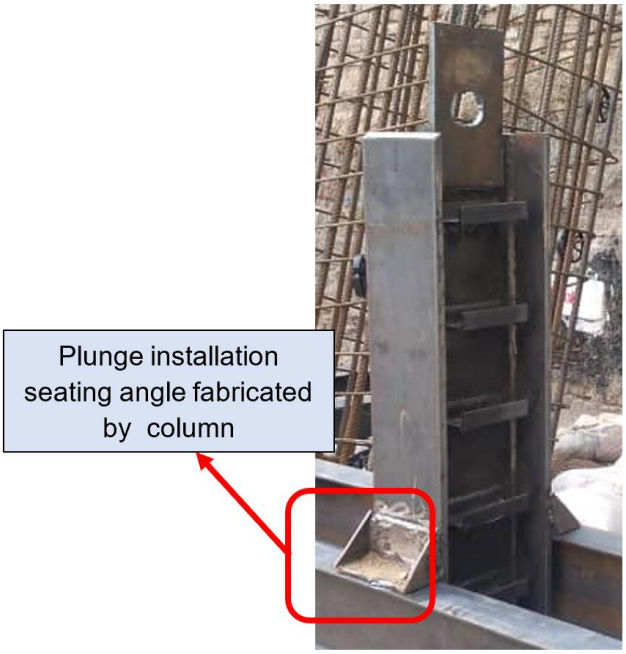


Figure 3.29 Prefabricated seating angles at the design level to facilitate plunge installation

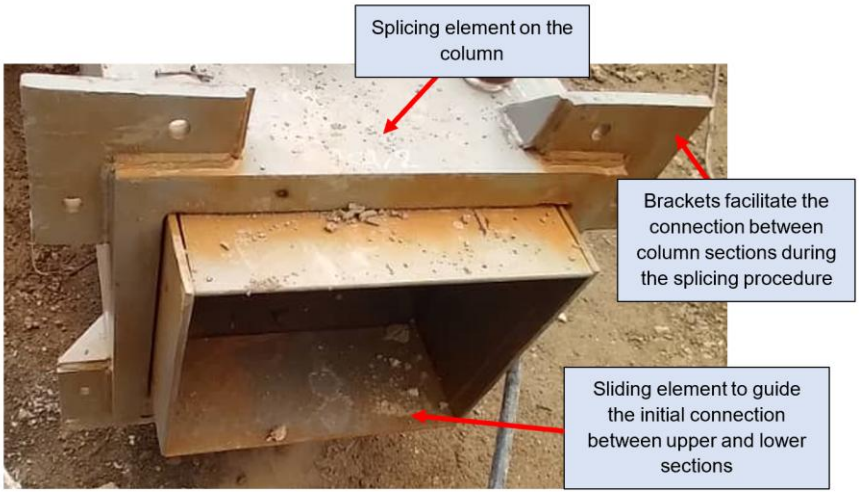


Figure 3.30 Prefabricated additional components to facilitate on-site splicing operations

The fabrication, connection, and installation of steel elements within steel-reinforced concrete and concrete-filled steel tube columns are similar to those of standalone steel columns.

Shear connectors are used to connect steel columns to retaining walls, shear walls, and other structural elements. When installing embedded columns in shafts, it is crucial to ensure that the shear connectors are compatible with the construction method. Figure 3.31 shows a column with welded head studs, where subsequent construction activities, such as soil and cement operations as well as excavation, have damaged the effectiveness of connectors. To mitigate this risk, careful design and construction practices must be implemented when using the top-down method to maintain the integrity of the shear connectors.



Figure 3.31 Damaged head studs on a column following soil and cement operations within the shaft and excavation

3.2.2.2.2.2 Construction and Installation of Concrete Vertical Load-bearing Components

Various approaches can be employed for the construction of concrete structures using the top-down method. Precast concrete columns, steel-reinforced concrete columns, and in-situ concrete

pouring within shafts are all viable options. The following sections will delve into the details of each approach.

3.2.2.2.2.2.1 Precast Concrete Columns

Precast concrete columns can be installed within shafts as prefabricated elements. While it's possible to fabricate these columns on-site, manufacturing them in a controlled factory environment is generally more efficient and reliable. Factory production allows for better quality control, reduced construction time, and minimizes the impact of site conditions on the production process.

The first stage of fabricating precast columns involves procuring a detailed list of cuttings and rebar ridges based on shop drawings. A shop drawing of columns should include the following information:

- Detailed dimensions of column cross-sections in all underground levels, preferably using consistent dimensions.
- Detailed configuration and placement of longitudinal reinforcement, ensuring adequate provision for starter bars for beams and walls.
- Precise dimensions and angles for beam-column connections.
- Detailed configuration of beam reinforcement within column cross-sections, including potential modifications to initial reinforcement designs where higher grade rebars are required.
- Detailed reinforcement configuration for shear walls and retaining walls within the column, including final wall dimensions and horizontal reinforcements within walls.
- If a fixed connection to the pile is required, the column design should include starter bars to enable anchoring to the pile.

- If shaft head frames are used for column installation, the column design should account for the required extension of longitudinal reinforcement beyond the column head (at least 15 cm).

Next, formworks are prepared to hold the longitudinal reinforcements in place. These frames are designed according to the reinforcement configurations and placement requirements and are positioned at each end of the element. Longitudinal reinforcements are inserted through the holes in the formworks, with the process continuing until all reinforcements are properly placed.

It is crucial to ensure that sufficient closed stirrups are positioned between each pair of formworks before the longitudinal reinforcements are installed. Once all longitudinal reinforcements are in place, the closed stirrups are secured and connected to the reinforcements, followed by the placement of single-legged stirrups. All placements must strictly adhere to the shop drawings.

After the reinforcements and stirrups are installed, starter bars are added to establish connections between beams, walls, and footings. Due to the precision required for installing these components, separate formworks made of cold foam are used for them. These formworks are placed in designated holes for each underground level, after which the insert reinforcements are installed within the frames.

Once column formworks and reinforcements are in place, side formworks can be installed. It is recommended to use single-piece formwork panels to prevent column deformations during concrete pourings. After formwork installation, concrete is poured and cured. Concrete mixture design should be based on properties outlined in shop drawings, keeping in mind that concrete used in precast columns should have a minimum grade of C30. Furthermore, mixture design and curing should be done so that the concrete reaches the initial strength required for moving

components as soon as possible. To achieve this, the following measures are recommended:

- Using a coherent distribution of aggregates and filler materials such as stone powder to improve workability.
- Using admixtures such as microsilica grout or gel, and superplasticizers to improve quality.
- Reducing the amount of water in the mixture by using superplasticizers to improve quality.

Figure 3.32 demonstrates on-site reinforcement for a precast concrete column and Figure 3.33 demonstrates reinforcement in prefabricated columns transferred to the project site.

The installation of precast concrete columns is similar to that of steel columns in that shaft head frames can be used to facilitate alignment and stabilization processes, as shown in Figures 3.17, 3.23, and 3.24. The column's axis is established at the shaft head and then, the column is aligned using the shaft head frame. Next, the column is connected to the pile by anchoring its reinforcement bars to the pile as described above. Once alignment is complete and the column is stabilized at the desired axis, pile concrete pouring operations can be carried out. When pile concrete is set, the steel frame at the shaft head can be removed. Given the fact that starter bars are already in place, precision is key in carrying out plunge and embedded installation.

Proper joint design is crucial for the structural integrity of prefabricated components. As a general rule, in-situ joints tend to be simpler and more robust compared to their prefabricated counterparts. In structures incorporating prefabricated components, joints should be located at points of minimal internal stress. This is especially important for structures located in seismic zones, where significant deformations may occur. In-situ concrete casting, post-tension cables, and high-strength bolts can be used for joining prefabricated components.

When using in-situ concrete casting to join prefabricated components, starter bars extending beyond the prefabricated elements are embedded in the concrete to provide continuity. Here, shear transfer is achieved through friction between the concrete elements, augmented by reinforcement bars perpendicular to the joint interface, while bending transfer is facilitated by compressive stresses in the concrete and tensile stresses in the reinforcement bars perpendicular to the joint interface.

In-situ concrete casting to join prefabricated components offers a simple and reliable connection which can also provide ductility if required. However, it can disrupt and delay the installation process due to the required curing time. Additionally, there may be visual discrepancies between the in-situ concrete and the prefabricated components, particularly in terms of color and texture. Furthermore, reinforcements used in some prefabricated components might be incompatible with reinforcements used in others.



Figure 3.32 Reinforcement in precast concrete columns

When using couplers to connect reinforcement bars in precast concrete elements or to extend concrete elements, it is essential to

ensure proper rebar preparation. Rebar ends should be cut using appropriate tools like reciprocating saws, circular saws, or angle grinders. The use of bolt cutters is not recommended, as they can damage the rebar. Additionally, cut threading the rebar is not acceptable, as it reduces the cross-sectional area of the bar and compromises its strength.



Figure 3.33 Prefabricated reinforced concrete columns

3.2.2.2.2.2.2 Construction and Installation of Composite Vertical Load-bearing Components

Steel-reinforced concrete (SRC) columns are composite members that combine the advantages of steel and concrete. During construction, the steel core primarily carries the loads, while the

surrounding concrete shares the load-bearing responsibility with the final structure. The initial step in constructing an SRC column is to embed the steel core within the shaft. The core can be configured as a box, H-section, tube, or cross-section. To enhance the axial load-bearing capacity, the core can be designed as a concrete-filled tube (CFT).

The installation of CFT columns also begins with embedding the steel tube within the shaft. To ensure a seamless concrete pour, the steel tube should have a continuous, closed cross-section, such as a box or tubular shape.

The construction and installation of steel elements within composite columns, such as SRC and CFT columns, follow similar principles to those outlined for standalone steel columns. For CFT columns, the steel tube is first installed within the shaft and the shaft is filled with appropriate materials, a process we will describe in detail in a future section. Subsequently, the tube is filled with concrete, ideally using a self-compacting concrete (SCC) mix to ensure proper workability.
The construction of concrete elements within composite columns is a complex process. In columns where the steel core is first installed within the shaft as a vertical load-bearing component, the column section is completed after excavation. Then, a concrete jacket is executed around the core to form a composite column. The steel core should be thoroughly cleaned to remove any soil or debris once excavation operations are complete. Pneumatic equipment, high-pressure water jets, or wire brushes can be used to clean steel cores. Next, reinforcement bars of the composite columns are placed around the core. The installation of starter bars and rebar connections, and the maintenance of couplers involve stages similar to retaining walls. Figures 3.34 and 3.35 demonstrate reinforcement around a steel core and formwork on a composite column, respectively. Figure 3.36 demonstrates a starter bars on the final cross-section of a composite column ready to complete the column for the next underground level. Using composite

vertical components is among the most suitable approaches to the construction of concrete structures in the top-down method.



Figure 3.34 Reinforcement around a steel core in preparation for concrete pouring on a composite column



Figure 3.35 Wooden and steel formwork on a composite concrete column



Figure 3.36 The final cross-section of a composite concrete column

3.2.2.2.2.2.2 Cast-in-place Concrete Columns

Two approaches can be adopted to construct cast-in-place concrete columns. The first approach is similar to what we saw in pile concrete pouring. After excavating the shaft and installing the pile reinforcement cage, concrete is poured to form both the pile and the column. It is essential to incorporate starter bar for future connections to beams, walls, and decks. Couplers can be used to splice reinforcement bars within the shaft, similar to what was described regarding precast concrete columns. However, precise reinforcement placement is crucial, as modifications may be limited due to the confined space. Rebar epoxy grouted techniques can be employed to embed reinforcement for future connections of deck, beam, or wall elements to the concrete column. Concrete pouring is carried out inside the shaft, and curing will involve some water reduction due to soil proximity. Furthermore, the concrete surface may exhibit a rough finish, as shown in Figure 3.37, due to irregularities in the shaft walls, necessitating

additional time and effort to furbish the concrete surface. Given these disadvantages, it is generally not recommended unless specific project circumstances dictate its use.



Figure 3.37 Cast-in-place concrete columns executed inside shafts for a top-down construction project in the United States

The second approach involves a staged construction process, where reinforcement, formwork, and concrete pouring are performed sequentially, starting from the pile head and progressing upwards. Here, similar to what was described about precast concrete columns, starter bars are embedded within the columns using couplers or through epoxy grouted to facilitate future connections. Accurate placement of these starter bars is crucial during both the design and construction phases. Figure 3.38 demonstrates an instance of this approach, where the shaft is stabilized with a steel casing, and reinforcement and formwork are being installed. Ensuring a safe working environment within the shaft is paramount. Adequate access, ventilation, lighting, and workspace, as depicted in Figure 3.38, are essential. Additionally, stabilizing the shaft walls is crucial to prevent collapse and maintain worker safety.

A structural engineer should be consulted to ensure proper design and inserting of connections. The connection details, such as the use of couplers or epoxy grouted, will depend on factors like the construction sequence and concrete casting method. It's essential

to consider the specific requirements and limitations of each approach to ensure the structural integrity of the connections. Figure 3.39 demonstrates an instance of this method, where starter bars for connecting beams to the column are inserted as couplers before concrete is poured. Similar to traditional bottom-up construction, this method involves sequential concrete placement, starting from the bottom of the shaft and progressing upwards. After each concrete pouring stage and formwork removal, the column is secured to the shaft wall to maintain its plumb.

Using SCC concrete is essential for these columns, and the concrete should be poured from the bottom up to minimize free-fall distances. Furthermore, this construction approach can be challenging due to the high-diameter excavation required and the potential difficulties in quality control. Therefore, this method is generally not recommended.



Figure 3.38 Reinforcement and formwork in a shaft stabilized through a steel casing; adequate access, ventilation, and lighting for workers

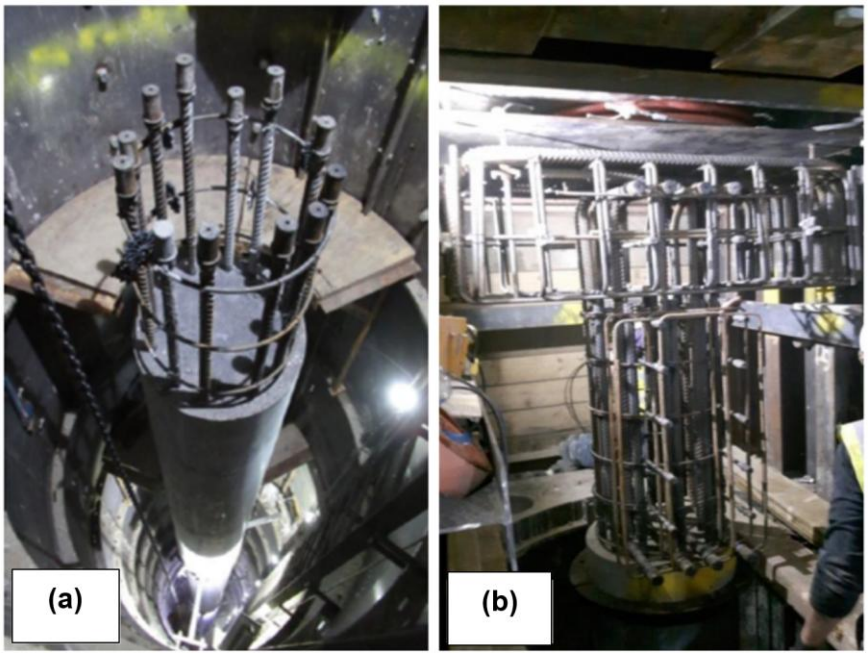


Figure 3.39 Bottom-up concrete column construction within a shaft: a) Sequential construction from bottom to top, anchorage of the column to the shaft wall; b) Inserting of starter bars via couplers to connect the column to a beam

3.2.2.2.3 Quality Control of Installing Vertical Load-bearing Components

Accurate alignment of the columns within the shaft is critical for the success of a top-down construction project. After excavation, ground leveling and regulation, and shaft head preparation, a surveying team should establish precise reference points at the shaft head (Chapter 5 will delve into surveying techniques in more detail). During column installation, these reference points can be transferred down the shaft using concrete pads or steel formworks, leveling lasers, and manual leveling instruments. To ensure accurate alignment, multiple reference points should be established along the column's length, allowing for visual inspection from outside the shaft. Additionally, checks from within the shaft can be performed using manual levels and leveling lasers. Once the column is aligned within acceptable tolerances, it can be secured in place.

3.2.2.3 Filling the Gaps between Shaft Walls and Vertical Load-bearing Components

Once vertical load-bearing components within shafts are installed using one of the methods mentioned above, the gaps between the components and the shaft walls must be filled with suitable materials. After installing columns and before installing beams and slabs in each basement level, when a considerable part of a column remains unanchored, it is essential to provide lateral support to prevent column deflection. The filling material should be sufficiently compact and durable to provide columns with enough confinement.

The selection of filling material depends on factors such as cost, availability, and the required level of compaction and confinement. Common options include angular sand, a screened mixture of site soil, a cement-soil mix, or low-cement concrete. Since these materials are typically poured from the top of the shaft, they should be free of large debris, such as cobbles to avoid damaging the column and its connections

Figure 3.40 illustrates the process of filling a shaft with a cement-soil mix. The soil is screened to remove large debris and then mixed with 100 kg of cement for each cubic meter of soil and some water to provide moisture. A mini loader is used to prepare the mixture and pour it into the shaft. While using manual labor is possible for this task, it is generally more time-consuming.

Figure 3.41 demonstrates an alternative approach, where low-cement concrete has been employed. Here, after excavation around each column, its concrete casing is crushed using the breaker module installed on a mini excavator. This provides ideal filling for shafts, but crushing cured concrete can be costly and time-consuming.



Figure 3.40 A mini loader fills a shaft using a mixture of cement, water, and screened soil



Figure 3.41 A breaker module installed on a mini excavator crushing a concrete casing to provide filling for shafts

3.2.2.4 Embedded Retaining Walls

Embedded retaining walls, which are constructed within the ground before other excavation work, can be implemented using various techniques, including sheet piles, contiguous piles, tangent piles, secant piles, diaphragm walls, and soil-cement walls.

3.2.2.4.1 Shaft Head Preparation for Wall Installation

To guide the excavation equipment and facilitate the installation of wall reinforcement, a guide wall is constructed at the shaft head for each wall section. As shown in Figure 3.42, this guide wall is typically a reinforced concrete wall, which can be 80-120 cm deep and 20-25 cm thick depending on whether the soil is cohesive or cohesionless. It helps maintain vertical alignment during excavation and provides a reference point for installing wall reinforcement. Figures 3.43 and 3.44 illustrate the use of a guide wall and concrete spacer to support the reinforcement cage and maintain the vertical position of the diaphragm wall section while keeping the wall plunged to enable the splicing of the next part. Concrete spacers are used to provide adequate concrete cover and facilitate the installation of reinforcement within the shaft.

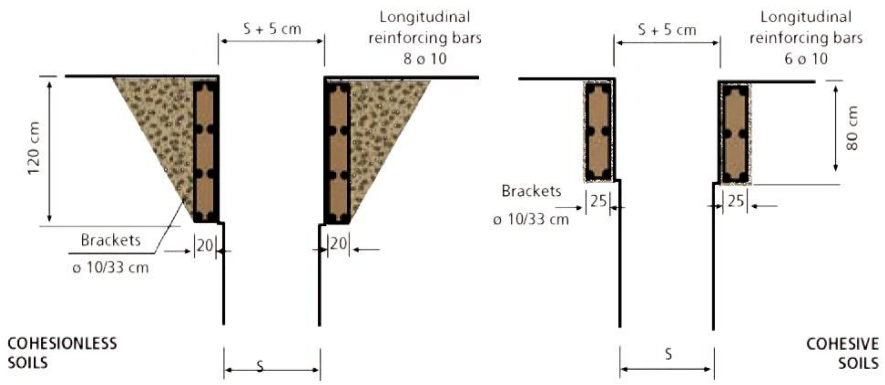


Figure 3.42 Guide walls used in cohesive and cohesionless soils to support a diaphragm wall

3.2.2.4.1.2 Pile Walls

When constructing cast-in-place concrete pile walls, steel casings can be used to stabilize the top few meters of the excavation in challenging soil conditions. These casings serve two primary purposes:

1. They prevent the collapse of the upper portion of the excavation. In soft soil conditions, the casing may extend the entire depth of the shaft.

2. They provide a vertical guide for reinforcement cage installation and facilitate splicing operations at the shaft head.

Figure 3.45 illustrates the installation of a reinforcement cage within a steel casing.

For secant piles, typically a cast-in-place concrete guide wall is installed on the ground level using a formwork made of polystyrene. This guide wall has the exact shape and dimensions of the secant pile. Once the guide wall is in place, it facilitates excavation and reinforcement cage installation. The guide wall helps keep the construction in line and guides the excavation equipment throughout their operations and helps keep the alignment for the vertical installation of the pile reinforcement cage. Figure 3.46 demonstrates an instance of formwork for a guide wall as well as a cast-in-place wall to install a secant pile wall.



Figure 3.43 A concrete spacer used to install a concrete wall



Figure 3.44 Guide walls used to install a diaphragm wall reinforcement cage while keeping the bottom part of the wall plunged at the shaft head to facilitate the splicing of the top part

For secant pile walls, a cast-in-place concrete guide wall is typically constructed at ground level using a polystyrene formwork. This guide wall has the same dimensions as the planned secant pile and serves as a reference for excavation and reinforcement cage installation. The guide wall helps maintain the correct alignment for both the excavation process and the vertical installation of the pile reinforcement cage while acting as a reference for excavation equipment. Figure 3.46 illustrates the

formwork for a guide wall and the resulting cast-in-place wall, which will be used to construct a secant pile wall.



Figure 3.45 Using steel to install a reinforcement cage



Figure 3.46 A concrete guide wall constructed using polystyrene formwork used for construction of a secant pile wall

3.2.2.4.2 Sheet Piles

Sheet pile walls are a well-established approach to constructing retaining walls. Sheet piles, manufactured in various shapes and sizes, are installed into the ground using specialized piling equipment including piling hammers. The interlocking design of tongues and grooves on sheet piles allows them to form a continuous wall. Piling equipment, such as vibratory hammers or impact hammers, can be mounted on cranes or excavators. This method eliminates the need for extensive soil excavation and removal.

Various waterproofing techniques can be applied to sheet pile walls to prevent water infiltration through the joints. Sheet piles can be used in a wide range of soil conditions, but their effectiveness may be limited in hard soils, as the required cross-sectional area may be excessive. In softer soils, sheet pile walls are a viable option. However, the noise and vibration associated with pile driving can be problematic in urban areas.

Sheet piles can be used as temporary retaining systems, which are removed once the permanent retaining wall is constructed. Alternatively, they can be used as permanent retaining walls in top-down construction, where they are installed before excavation operations begin. Sheet piles can also be used to transfer gravity loads.

Figure 3.47 demonstrates the driving of steel profiles of a sheet pile using vibratory hammers which can be installed on excavators and cranes. Figure 3.48 shows a sheet pile wall used as a permanent retaining element in top-down construction.

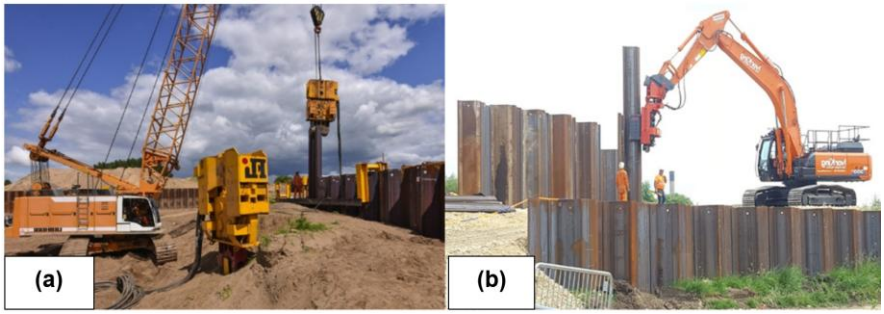


Figure 3.47 Driving sheet pile sections through the soil using piling hammers: a) piling hammer mounted on a crane; b) piling hammer mounted on an excavator



Figure 3.48 Using sheet piles as permanent retaining walls in a top-down construction of a multi-level car park

3.2.2.4.3 Pile Walls

Pile walls can be constructed using various configurations. In some cases, there is a space between piles (contiguous walls); in other cases, they touch each other (tangent walls); and in still others, they cross each other (secant walls). Furthermore, multiple layers of piles can be constructed next to each other. Figure 3.49 demonstrates a cross-sectional view of these pile configurations.

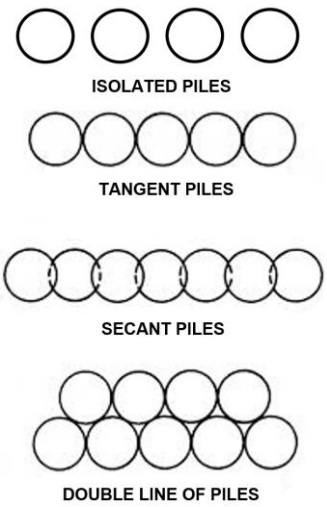


Figure 3.49 Cross-sections for various pile wall types

Contiguous pile walls can be constructed using manual excavation, cast-in-place concrete, or mechanized methods. A common mechanized method involves the use of a continuous flight auger (CFA) to create the excavation. The distance between adjacent piles is typically 1.5 to 3 times the pile diameter. After excavation, reinforcement cages are installed, followed by concrete pouring (Figure 3.50). This retaining method is suitable for soil conditions with low permeability, such as clay soils, where managing soil dispersion is necessary to control underground water seepage. However, it may not be ideal for highly permeable, soft soils, like sandy soils. Figure 3.51 shows a cross-section of a contiguous cast-in-place concrete pile wall. Figure 3.52 illustrates a contiguous wall in a top-down construction project. While contiguous walls should be only used in stable soils, tangent and secant pile walls can be used in most soil conditions, as long as suitable tools and equipment are used. These walls are embedded and constructed before excavation. After the excavation, they can be sequentially covered with a permanent concrete retaining wall from top to bottom as underground construction operations are underway.

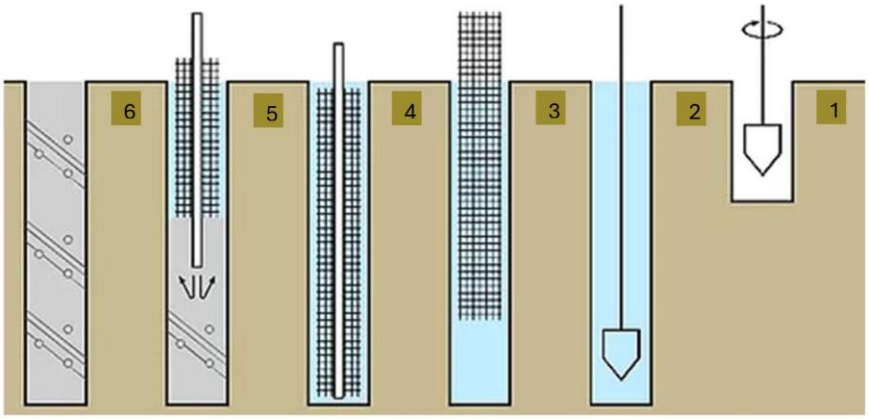


Figure 3.50 Cast-in-place concrete pile construction stages: 1) Excavation; 2) Stabilizing shaft walls using bentonite if necessary; 3) Installing the reinforcement cage (or steel profiles); 4) Positioning the tremie tube within the shaft; 5) Bottom-up pile concrete pouring using the tremie tube; 6) The completed cast-in-place concrete pile



Figure 3.51 Side view and cross-section of a contiguous pile wall showing the distance between piles, necessitating soil stabilization



Figure 3.52 Contiguous pile wall as an embedded retaining element in a top-down construction project. After excavation and as underground operations are underway, the pile wall is covered with a permanent concrete retaining wall from top to bottom.

For tangent and secant pile walls, excavation is carried out using excavators. Next, reinforcement cages or steel sheets are installed inside shafts. Then, concrete is poured using tremie tubes (Figure 3.50). Pile reinforcement can be installed as steel profiles or rebars. Concrete piles executed using this method are interlocked to form a continuous concrete wall.

In secant pile walls, piles are usually executed as alternating hard-soft (in most cases) and hard-hard elements next to each other. Various configurations of secant piles are demonstrated in Figure 3.53. First, soft piles are executed using a soft concrete mix; these piles are not reinforced. Next, hard piles are executed using structure concrete next to the soft pile. In the hard-hard variation, pile reinforcements are configured so that they are outside the interlocking area between two piles. Hard piles can procure all the resistance and rigidity a structure needs as they are reinforced elements. A portion of the first cast piles is chafed by the excavator to make the necessary overlap area between the two piles. As long as related quality and layering tolerances are taken into account, tangent pile walls can work as a temporary grout curtain. For a

permanent solution, wall grouting considerations must be taken into account according to related regulations.

Figure 3.54 shows the construction of an embedded tangent pile wall in a top-down project. The retaining wall is constructed sequentially as construction progresses in underground levels, with each section of the wall being supported by the underlying pile wall.

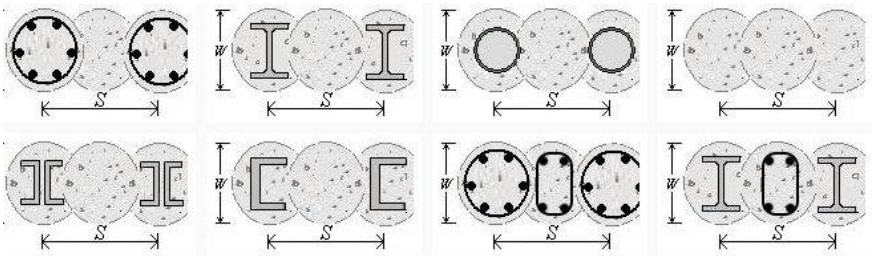


Figure 3.53 Secant pile types: soft-hard piles, hard-hard piles; using steel profiles and reinforcement configurations in various forms

Compared to sheet pile walls and some other construction methods, pile walls offer several advantages, including increased rigidity, reduced noise pollution during construction, faster construction time, lower costs, and improved waterproofing performance. However, maintaining precise alignment tolerances for deep pile walls can be challenging. Additionally, non-interlocking pile walls may experience water seepage in water-bearing soils. Furthermore, contiguous pile walls may not be suitable for soft soil conditions.

3.2.2.4.4 Diaphragm Walls

Diaphragm walls are typically deep, embedded, cast-in-place concrete elements. To construct a diaphragm wall, first, a trench should be excavated using a crane-mounted grab or a milling cutter. This trench is then stabilized with a bentonite or polymer grout. Once the trench is excavated, a reinforcement cage is installed, and the trench is filled with concrete, displacing the grout. Diaphragm walls are constructed sequentially, with each

section connected to the previous one, forming a continuous, integrated structure (Figure 3.55).



Figure 3.54 Embedded construction of a tangent pile wall in a top-down construction project. The permanent retaining wall is constructed sequentially once excavation is over and underground construction operations are underway.

Diaphragm walls typically range from 40 to 150 centimeters in thickness and 250 to 700 centimeters in length. They are suitable for deep underground structures, such as metro stations, tunnels, and pumping stations. The technology can be used in confined urban spaces, where construction adjacent to existing structures is necessary. Diaphragm walls are used as retaining elements in conjunction with various stabilization techniques, including anchoring, nailing, and top-down construction.

Concrete diaphragm walls are rigid structures that can be used to control soil deformation and provide waterproofing. In some cases, they can also be used as load-bearing elements, transferring loads to deeper soil layers. These load-bearing diaphragm walls are often referred to as barrettes.

Poorly designed or incorrectly constructed diaphragm walls can significantly impact the overall project performance and cost-effectiveness. Therefore, careful structural and geotechnical analysis is essential.

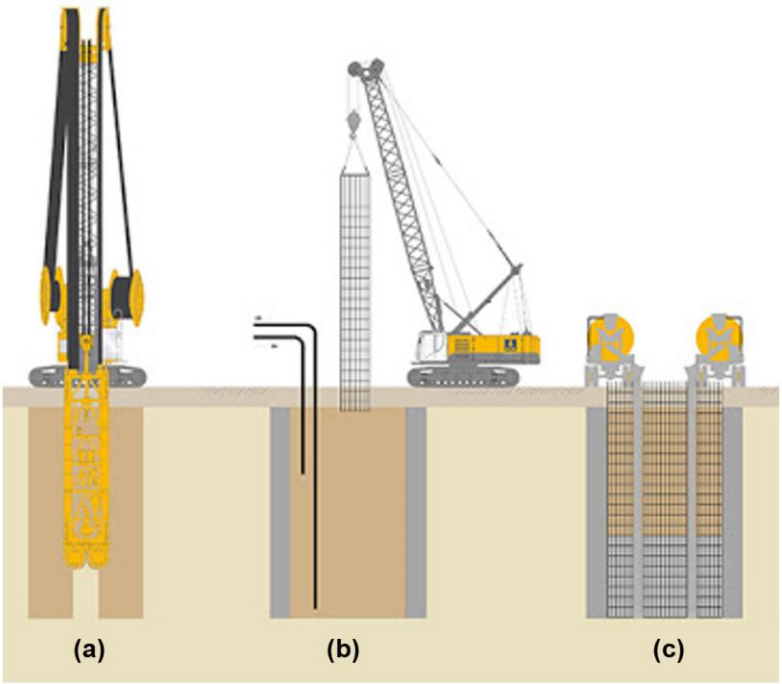


Figure 3.55 Diaphragm wall construction stages: a) Trench excavation and drilling mud circulation; b) Reinforcement cage installation inside the trench; c) Concrete pouring as drill mud is displaced

Diaphragm wall Construction stages:

- Before excavation begins, a precast or cast-in-place concrete guide wall is installed (Figure 3.56). This initial excavation, typically 1-1.5 meters deep, stabilizes the soil and provides a reference point for the excavation equipment. The guide wall facilitates the installation of the reinforcement cage and enables splicing operations at the trench head.

Additionally, using hydraulic rams, the guide wall can aid in the removal of end-stop plates. The distance between two guide walls on each side of a trench provides the space needed for drilling mud or bentonite-stabilized liquid.

- Next, individual panels are excavated using excavator grabs or hydromills to the desired depth (Figure 3.57). The choice of excavation equipment depends on the specific soil conditions. During excavation, bentonite slurry is used to stabilize the soil walls. The excavation should be as vertical as possible, with a maximum allowable deviation of 1%.
- Once excavation for one panel is over, end-stop plates are installed on each side of the panel to divide the concrete pouring stages. These plates, which have the same width as the panel, are removed once concrete is cured. The interlocking design of the end stop plates, often featuring one to three joints, allows for the installation of waterstop bands. Half of the waterstop band is embedded in the concrete of one panel, while the other half is embedded in the adjacent panel, creating a waterproof seal. Proper installation of these end-stop plates is crucial for the overall water tightness of the diaphragm wall. Figure 3.58 illustrates an instance of the installation of end stop plates and their waterstop bands.
- After excavation, the bentonite slurry is recycled and replenished to maintain its properties. The reinforcement cage is then installed within the trench (Figure 3.59), followed by concrete placement using a tremie pipe (Figure 3.60). During concrete placement, bentonite grout circulation and refreshment continue.



Figure 3.56 Guide wall preparation at the trench head before excavation operations

In cases where extra bending capacities are needed for the wall, T-shaped and cross-shaped sections can be used. To execute walls with these sections, a trench with a suitable shape is excavated and the reinforcement cage of an appropriate form is installed within the trench before concrete is poured. Figure 3.61 demonstrates T-shaped and cross-shaped wall trenches. Figure 3.62 illustrates the installation of a reinforcement cage as well as the final T-shaped section of a wall.



Figure 3.57 Trench excavation for a diaphragm wall using a grab and guide wall at the trench head with drill mud circulation

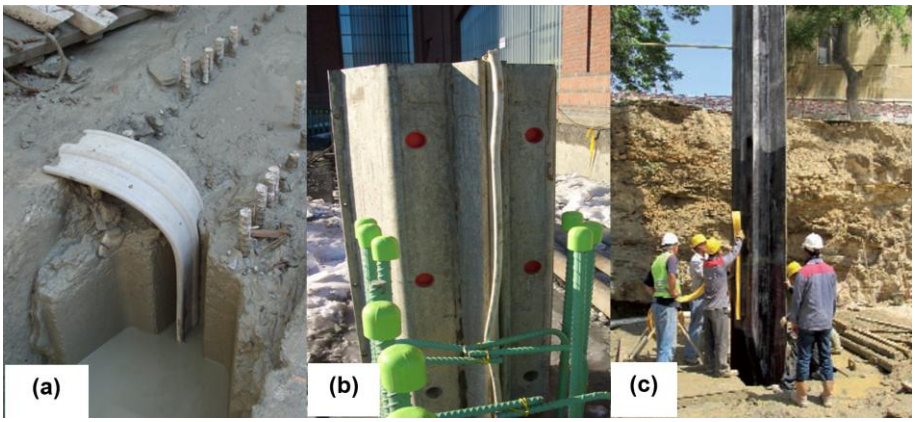


Figure 3.58 a) Installation of end stop plates in the trench; b) Installation of waterstop bands within the end stop plate joints; c) Waterstop band within a concrete wall after casting, ensuring a watertight seal between panels

Concrete diaphragm walls are one of the most common elements in top-down construction projects due their inherent advantages, such as mechanized construction suitability in urban environments where there are many adjacent buildings; minimal noise and vibration, making them ideal for urban settings; adaptability to various geotechnical conditions; potential to use as permanent structural elements; excellent water resistance; suitability for deep underground structures; potential to use as load-bearing elements; excellent rigidity; and minimal excavation-induced deformations.

However, there are some disadvantages engineers should keep in mind when using diaphragm walls, including design and execution uncertainties, the risk of trench wall instability during concrete placement, potential water tightness issues at joints, high construction costs, and significant site space requirements to position equipment and machinery.
Trench wall instabilities may occur during concrete placement, resulting in indentations or protrusions. Such defects must be addressed immediately. Additionally, the contact between the concrete and the soil can lead to surface imperfections,

necessitating subsequent furnishing operations. Figure 3.63 illustrates some common construction issues that may arise during diaphragm wall construction, which become apparent only once the excavation is over.

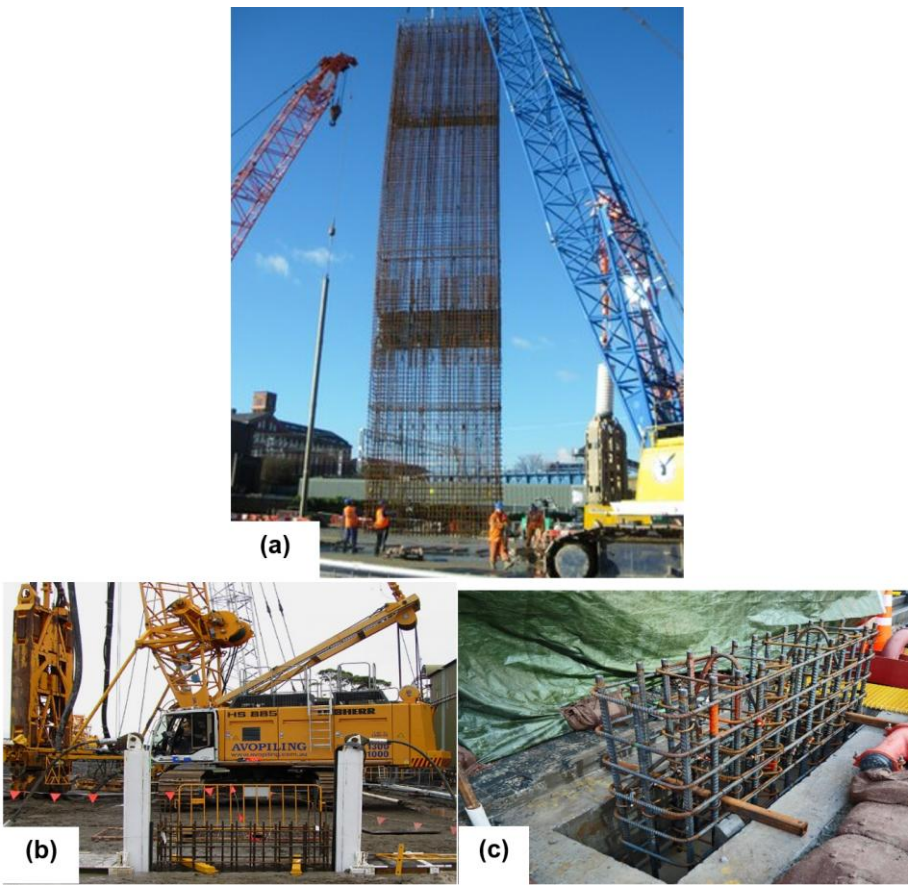


Figure 3.59 Concrete wall reinforcement cage installation: a) Placement of the reinforcement cage inside the trench using a crane; b) End stop plates on both sides of the trench, with the reinforcement cage positioned between them; c) Reinforcement cage plunged at the trench head, allowing for splicing operations facilitated through guide walls



Figure 3.60 Tremie tubes are used to carry out concrete pouring for wall panels



Figure 3.61 T-shaped and cross-shaped trenches excavated using excavator grabs and hydromills

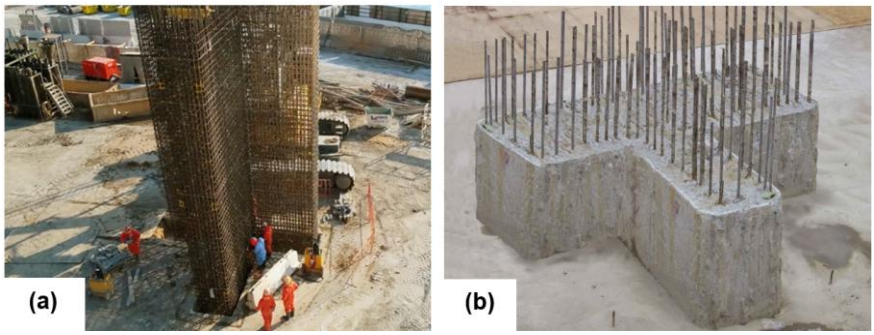


Figure 3.62 Execution of T-shaped diaphragm walls: a) Installation of the T-shaped reinforcement cage; b) The final T-shaped section of the wall

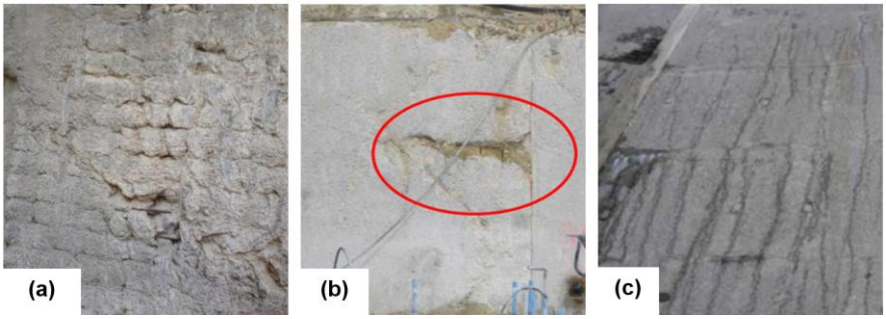


Figure 3.63 Examples of construction issues that may become apparent after excavation is complete: a) Protrusion of reinforcement bars pointing to the insufficient concrete flow in the empty spaces between rebars and the surrounding soil; b) A wall indentation caused by soil instability or the presence of rocks in the trench wall; c) Concrete bleeding appearing as linear watermarks

3.2.2.4.5 Soil-cement or Soil-crete Walls

Soil-cement and soil-crete walls are in-situ diaphragm walls. Their construction does not involve excavation. These walls are formed by mixing the in situ soil with cement grout. Two primary methods are used to construct soil-cement walls: deep soil mixing, where soil is mixed with cement grout using a rotating auger or cutter; and jet grouting, where high-pressure jets of cement grout are injected into the soil to create a soil-cement mixture. Figure 3.64 illustrates the principles of soil-cement wall construction. While their materials and construction methods differ from traditional diaphragm walls, soil-cement walls offer similar performance. However, it is important to note that soil-cement walls typically have a lower load-bearing capacity.

As mentioned above, the construction of soil-cement columns does not involve soil removal and there is no need for excavation. In cases where significant rigidity and bending capacity are necessary, these columns can be reinforced using steel profiles or rebars (Figure 3.65).

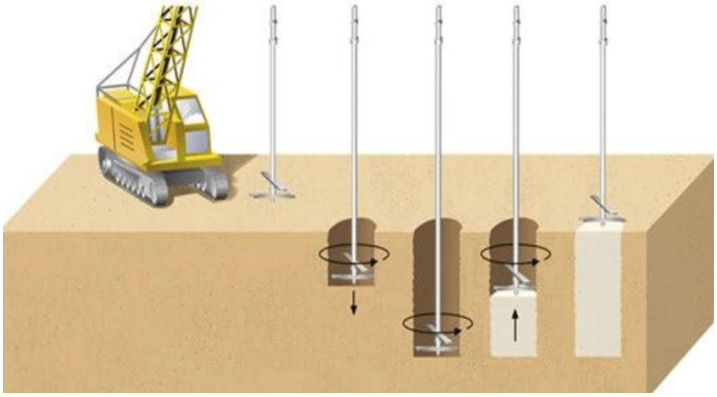


Figure 3.64 A schematic view of deep soil mixing



Figure 3.65 Adding bending capacity to a soil-cement wall by driving I-shaped steel profiles into them and installing rebars within the mixture before the cement is set

The same construction procedure can be used to create columns with various cross-sectional shapes. Figures 3.66 and 3.67 show some examples of these columns.

Soil-cement walls offer several advantages, such as utilizing on-site soil, eliminating the need for excavation, minimizing noise and vibration, and applicability in high groundwater conditions. They also provide watertightness and strong resistance to water seepage.

Soil-cement walls are not commonly used in top-down construction due to their low load-bearing capacity. However, they can help stabilize shallow shafts in soft soil conditions when combined with other methods, such as anchoring. Furthermore, in top-down construction, where peripheral walls must support gravity loads, soil-cement walls are generally unsuitable. As soil-cement technology continues to evolve, ongoing research may lead to advancements and broader applications in the future.



Figure 3.66 A deep soil mixing machine with three drills used to mix soil and cement grout; soil-cement columns comprised of three overlapping columns



Figure 3.67 A cutter deep soil mixing machine that mixes soil and cement vertically; the final soil-cement column with a rectangular cross-section

3.2.3 Underground Earthworks

The efficiency of earthworks significantly impacts the overall success of top-down construction projects. As work progresses beneath multiple floor slabs, excavated soil must be removed through openings in sides or ceilings. The complex interplay between various structural elements can hinder continuous excavation, making the optimization of soil removal processes a critical consideration for industry experts.

Top-down construction involves a three-stage excavation process, each requiring specialized techniques and equipment. The first stage involves soil excavation in basement floors within confined spaces where many structural components may be present. The second stage focuses on transferring the excavated soil through openings in sides or ceilings. The final stage involves hauling the soil to the ground level where it is loaded on trucks and removed from the site.

3.2.3.1 Soil Excavation

Underground soil excavation can be carried out using mini, midi, or standard excavators. Selecting the appropriate excavator for underground excavation depends on various factors, including soil conditions, site dimensions, available workspace (headroom and column spacing), and the required machine power.

The selection of the excavator type is influenced by its spatial requirements—including arm height, length, width, and turning radius—as well as excavation depth. These factors, along with site dimensions, column spacing, and available headroom, must be considered during the design phase. If sufficient headroom and site dimensions are available, standard excavators can be used for underground excavation. More powerful excavators can expedite the excavation process. In challenging soil conditions, smaller excavators may not be sufficient unless they are equipped with breaker attachments. This can slow down the excavation process. In such cases, midi or standard excavators may be more suitable.

Figure 3.68 provides an example of dimensions provided by excavator manufacturers. These dimensions can be used to assess the available workspace and select suitable excavation equipment.

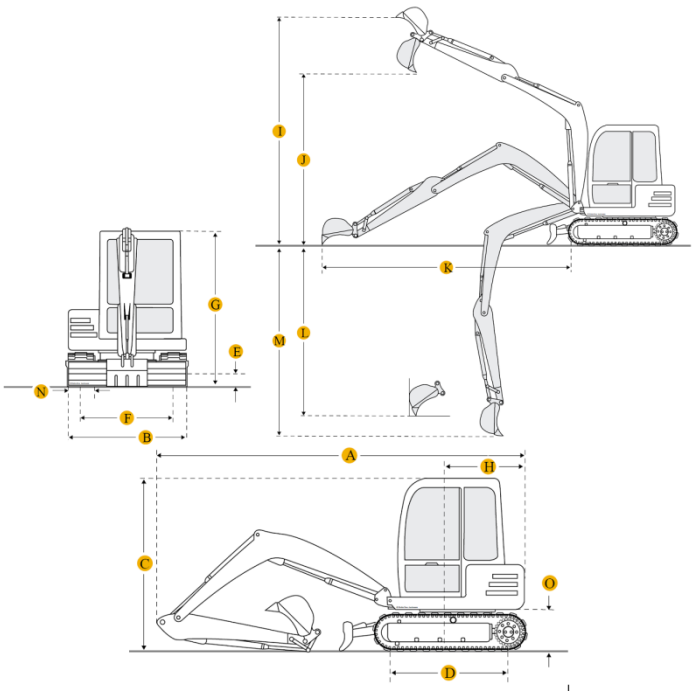


Figure 3.68 Excavator dimensions provided by a manufacturer, presenting details essential for selecting suitable underground excavation equipment

Next, we will examine each excavator type in detail.

3.2.3.1.1 Mini or Compact Excavators

Mini excavators, the smallest class of excavators, are well-suited for confined spaces. Their compact size and light weight make them highly maneuverable in the limited underground environments typical of top-down construction. Mini excavators come in various models, often equipped with sophisticated auxiliary equipment.

These machines typically have a working weight of 1 to 10 tonnes and can excavate to depths of 1.7 to 5.2 meters. Their bucket capacity ranges from 0.014 to 0.28 cubic meters. When selecting a mini excavator, it is essential to compare manufacturer-provided specifications against site dimensions and available workspace. Figure 3.69 illustrates a mini excavator well-suited for the column spacing and headroom constraints of a particular construction site.



Figure 3.69 A mini excavator operating at a top-down construction site

3.2.3.1.2 Midi Excavators

Midi excavators offer increased power and capacity compared to mini excavators. They typically weigh up to 38 tonnes and have a bucket capacity of up to 1.5 cubic meters. While midi excavators require more workspace and have a larger turning radius compared to mini excavators, they offer significantly higher excavation capacity, reaching depths of approximately 8 meters. Figure 3.70 illustrates a midi excavator operating at a top-down construction site.



Figure 3.70 Underground excavation by a midi excavator at a top-down construction site where adequate headroom and column spacing are available

3.2.3.1.3 Standard Excavators

Standard excavators weigh between 7 to 45 tonnes and have a bucket capacity of up to 2.5 cubic meters. While they offer faster excavation, standard excavators require an unconstrained workspace and larger ceiling openings for transportation due to their larger size. Figure 3.71 illustrates a standard excavator operating at a top-down construction site where the floor slab remains incomplete to facilitate equipment maneuverability. In this example, each excavation stage involves removing soil from two floors. Careful planning is essential to ensure compliance with design assumptions, and the top-down construction advisor engineer must be consulted before standard excavators can be utilized at a site.

3.2.3.2 Soil Transfer

Once excavation is complete, the removed soil must be transferred to pre-installed ceiling openings. This is typically accomplished using equipment such as loaders, mini-loaders, excavators equipped with telescopic arms (to enable horizontal movement), or conveyor belts. When selecting soil transfer equipment, factors such as site dimensions, equipment capacity, and efficiency must be taken into account.



Figure 3.71 Simultaneous excavation in two underground floors, with adequate workspace for large equipment

3.2.3.2.1 Loaders and Mini-loaders

Loaders and mini-loaders are commonly used for transferring soil and other materials in underground top-down construction. As with other equipment types, careful consideration must be given to site dimensions and equipment capacity. Loaders are typically more efficient than mini-loaders due to their higher capacity. However, their larger size means their transfer and positioning require bigger site dimensions and opening sizes. Mini-loaders, on the other hand, are smaller and better suited for work in smaller, more constrained sites typical of top-down construction, but their limited capacity can hinder a project's progress. Mini-loaders have a bucket capacity of 0.3 to 1.6 cubic meters and a working weight of between 1.5 to 6 tonnes. Loaders' bucket capacity can be anywhere between 0.8 to 32 cubic meters and their working weight can be between 3 to 65 tonnes. Figure 3.72 shows a mini-

loader and a loader operating underground at a top-down construction site.

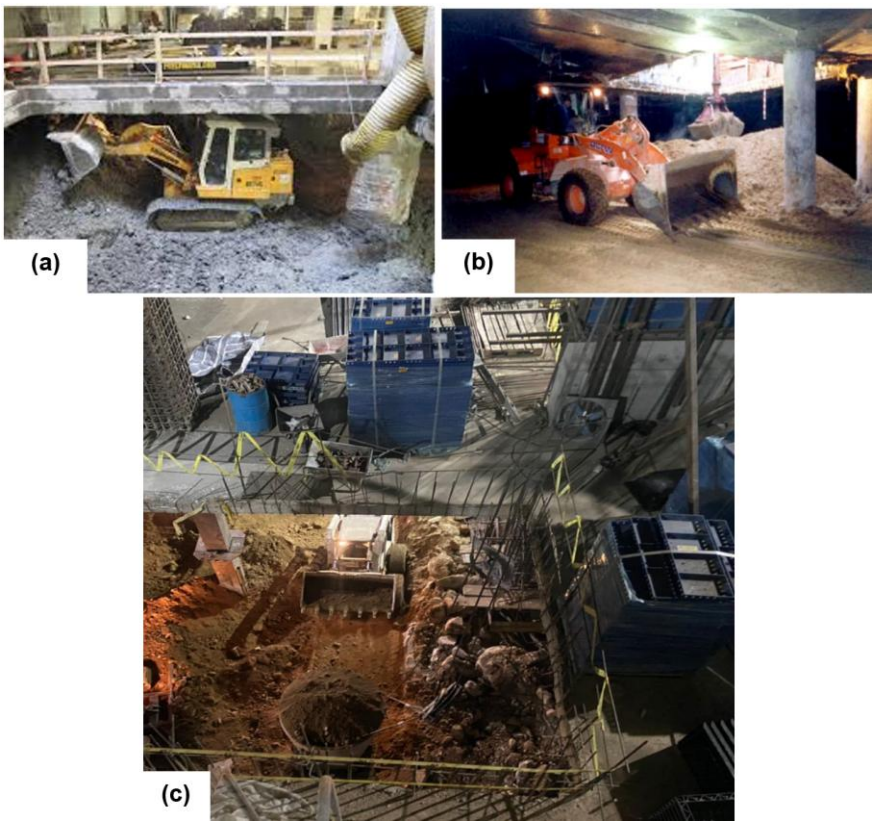


Figure 3.72 a) A mini-loader operating at a small, constrained underground site, part of a top-down construction project; b) A loader transferring soil to an opening at an underground level of a top-down construction site

3.2.3.2.2 Excavators Equipped with Telescopic Arms

Excavators equipped with telescopic arms can be used to transfer soil within underground top-down construction sites. These arms enable horizontal movement while the excavator's relatively low profile allows for operation under low-clearance floor slabs. Figure 3.71 shows an excavator equipped with a telescopic arm transferring soil horizontally to a depot position through an opening in a low-ceiling underground area.



Figure 3.73 Horizontal soil transfer in a low-ceiling underground area at a top-down construction site using an excavator equipped with a telescopic arm

3.2.3.3 Soil Removal and Disposal

After excavation, soil must be removed from the site. Various methods and equipment, including excavators, telescopic-boom excavators, tower cranes, lattice-boom crawler cranes, hydraulic mobile telescopic-boom cranes, gantry cranes, conveyor belts, and lifts, can be used for soil transfer. When selecting equipment, factors such as efficiency and the required dimensions of ceiling openings must be considered. For example, long-boom excavators require significantly larger working spaces and ceiling openings compared to telescopic-boom excavators. These considerations should be integrated into the structural design phase to ensure adequate ceiling openings are provided.

3.2.3.3.1 Standard Excavators

While primarily used for excavation, excavators can also be employed for soil removal, particularly for depths of up to 8 meters. Figure 3.74 illustrates a standard excavator removing soil through a ceiling opening at a top-down construction site. Some standard excavators, like the one shown in Figure 3.75, feature low-profile, circular cabins with a reduced turning radius, making

them more suitable for confined spaces. Ceiling openings must be designed to accommodate the excavator's operations.



Figure 3.74 Soil removal by a standard excavator through a ceiling opening

3.2.3.3.2 Long-boom Excavators

Long-boom (or long-reach) excavators feature extended arms, enabling them to excavate at greater depths, often up to 21 meters. However, most models available in Iran typically reach depths of around 15 meters. Their longer arms limit their bucket capacity to 0.5 to 1.5 cubic meters, which is generally lower than standard excavators. Before using long-boom excavators in top-down construction, careful consideration must be given to workspace requirements, turning radius, and arm extension length. These excavators typically require larger ceiling openings. For example, a long-boom excavator excavating to a depth of 15 meters may require an opening approximately 8 meters long (along the arm's axis) and 3 meters wide (perpendicular to the arm). An example of soil removal operations using a long-boom excavator at a top-down construction site is presented in Figure 3.76.



Figured 3.75 A specialized excavator suitable for operating in constrained spaces

3.2.3.3.3 Clamshell Telescopic Arm Excavators

Clamshell telescopic arm excavators feature a telescopic arm with a clamshell bucket, making them well-suited for removing soil from deep, confined spaces typical of top-down construction. The arm can extend up to 40 meters, and the bucket capacity can reach 1.5 cubic meters. Compared to long-boom excavators, clamshell telescopic arm excavators require less workspace due to their lower turning radius and single-axis arm operation. This can help preserve the structural integrity of floor slabs. Figure 3.78 illustrates a clamshell telescopic arm excavator in operation.

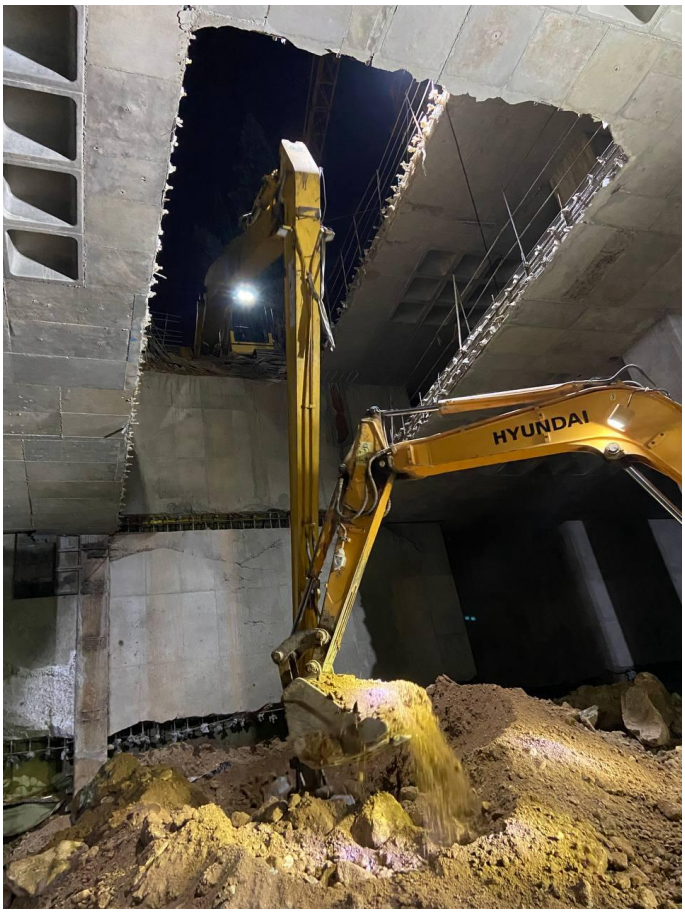


Figure 3.76 Soil removal from excavation site using a long-boom excavator

3.2.3.3.4 Tower Cranes

Tower cranes can be used to remove soil from top-down construction sites. However, careful planning is necessary to optimize their placement and the location of ceiling openings to maximize efficiency. Tower cranes, while suitable for certain applications, can be less efficient than other equipment due to their limited power and slower movement. Additionally, their capacity is influenced by the distance between their base and the excavation site and depot. Figure 3.79 shows a tower crane removing soil from a top-down construction site. The soil is deposited at exit points using mechanical grabs or self-unloading buckets

suspended from the crane. It is necessary to design ceiling openings so that they can accommodate these modules.

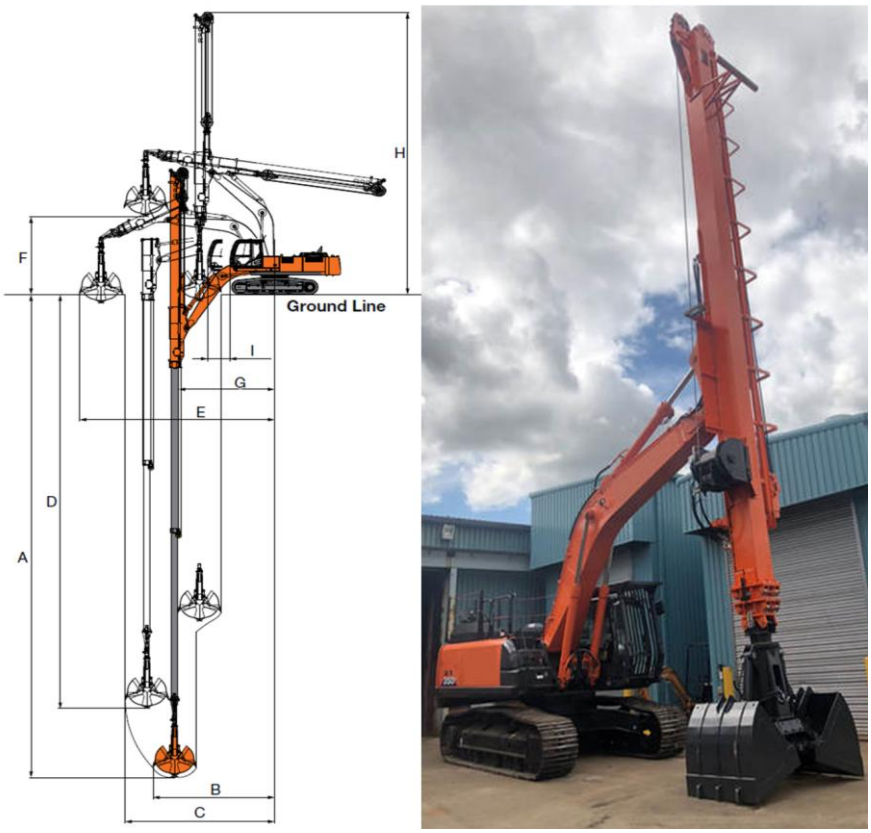


Figure 3.77 An excavator with a telescopic arm equipped with a clamshell bucket; the telescopic arm can extend on the vertical axis

An important consideration when using tower cranes at top-down construction sites is choosing the location where the crane is positioned. Engineers have three options in this regard:

1. The crane's base can be positioned behind the excavation wall, in a courtyard, or in an adjacent lot. However, the superimposed load from the crane must be considered in the design and analysis of the excavation wall's stability.

2. The crane's base can be positioned within the site at the same level as the structure's foundation or a level below. In this case, it is necessary to excavate a suitable shaft to accommodate the crane's specifications and its footing.
3. In cases where the first two options are not feasible, the crane cannot be used to install columns within shafts, unless it is first positioned with column installation operations in mind at the lot's natural level, and then dismantled and reassembled on the first-floor slab once it is in place. This approach is often impractical due to cost considerations, necessitating the use of mobile cranes for column installation.

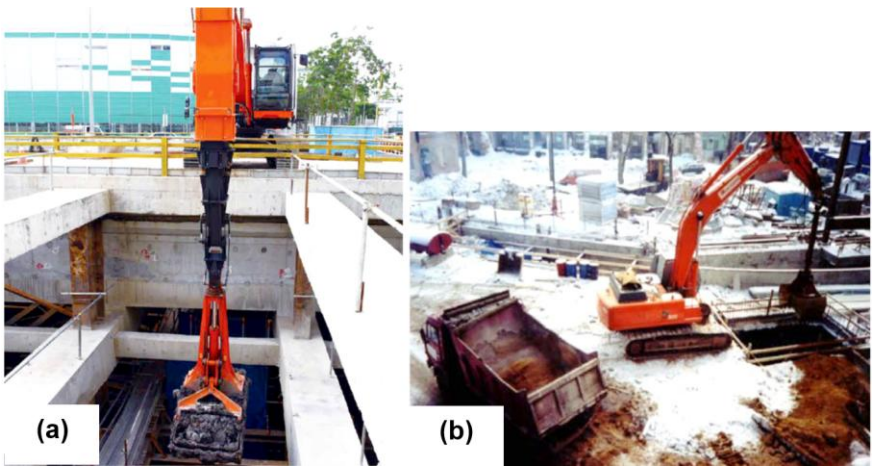


Figure 3.78 a) An excavator with a telescopic arm removing soil from a top-down construction site; b) the small ceiling opening required can be seen



Figure 3.79 Soil removal using a grab (a) and a self-unloading bucket (b) suspended from a crane

3.2.3.3.5 Lattice-boom Crawler Cranes

Unlike telescopic hydraulic cranes, lattice-boom crawler cranes, also known as dry-boom crawler cranes, use a fixed boom or mast. These cranes can be used for soil removal in top-down construction. They can be equipped with a grab or a self-unloading bucket. They come in various sizes and capacities, with some models capable of lifting several thousand tonnes at heights of up to 140 meters. Standard lattice-boom cranes used in Iran for soil removal typically have a nominal capacity of around 30 tonnes. These cranes are commonly used for soil removal in deep excavations exceeding 13 meters, where other equipment may be less effective or impractical. Figure 3.80 illustrates a schematic view of a lattice-boom crawler crane with attached grab and self-unloading bucket modules. Figures 3.81 and 3.82 show lattice-boom crawler cranes removing soil from excavation sites.



Figure 3.80 Schematic view of lattice-boom crawler crane with grab and self-unloading bucket



Figure 3.81 Soil removal from an excavation site by a lattice-boom crawler crane equipped with a soil removal container

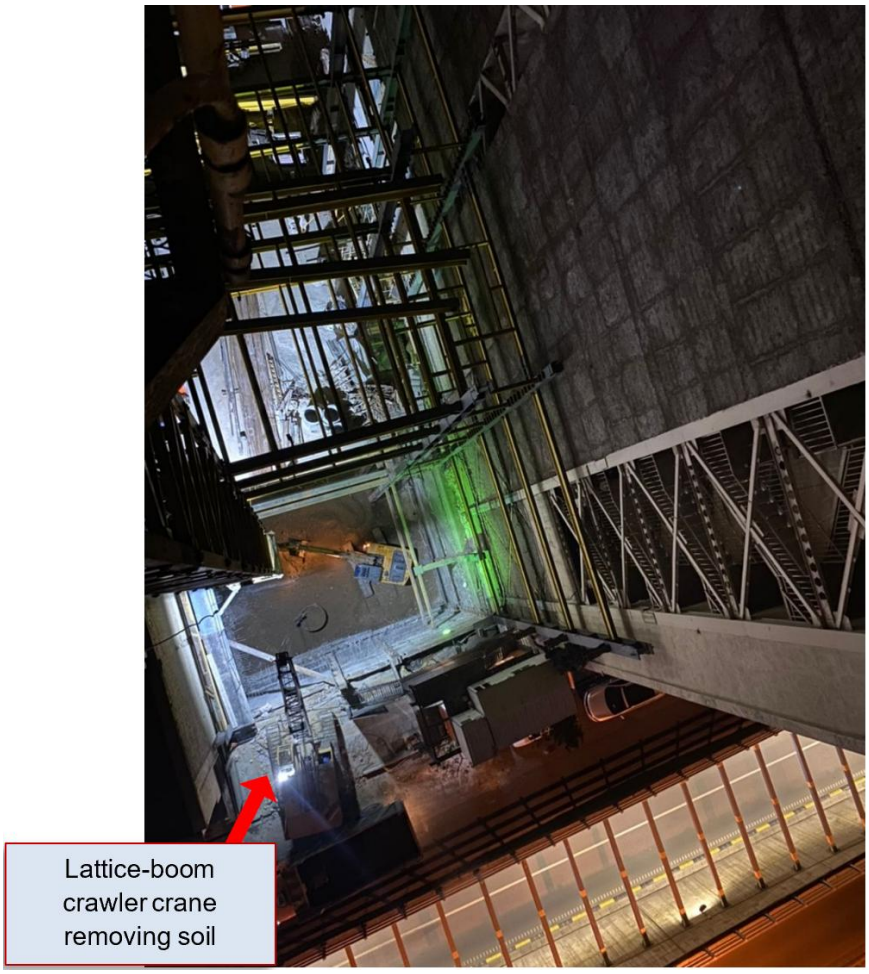


Figure 3.82 Lattice-boom crawler crane removing soil

3.2.3.3.6 Telescopic Hydraulic Cranes

Telescopic hydraulic cranes are among the most commonly used crane types and come in various capacities. While they can perform soil removal operations similar to lattice-boom crawler cranes, they are less frequently used for this purpose due to cost considerations. However, they remain a technically viable option.

3.2.3.3.7 Gantry Cranes

Gantry cranes, available in rail-operated and wheel-operated varieties, offer unique capabilities for soil removal operations. They can operate efficiently even under heavy loads. When using

gantry cranes, openings positioned near shaft walls should allow for truck access to facilitate soil removal. Due to their unique design, the crane's rails or wheels can be integrated into the structure itself, but this requires careful their careful integration into the structural design. Gantry cranes are widely used in metro station construction projects in Iran. Figure 3.83 shows a high-capacity gantry crane equipped with a grab with enough space under it to position a truck. Figure 3.84 shows a gantry crane transferring a container of oil-polluted soil.



Figure 3.83 Using a gantry crane for soil removal at a top-down construction site



Figure 3.84 Using a gantry crane for lifting and transferring a container of oil-polluted soil at a top-down construction site

3.2.3.3.8 Conveyor Belts and Lifts

In addition to transferring soil on the horizontal axis under floor slabs, conveyor belts can be used for transferring soil off the excavation site through ceiling openings. Their main advantages include high capacity and speed, as well as their more manageable opening requirements. However, they have some limitations as well, including their fixed operating angles, the potential need for multiple or intersecting belts (Figure 3.85), and the challenges associated with installation and space requirements. Designing and installing conveyor belts is a specialized task.



Figure 3.85 Transferring excavated soil from the site through floor slab openings using conveyor belts at a top-down construction site

Another option for soil removal is the use of lifts, which can efficiently transport soil vertically. Lifts offer the advantage of requiring relatively small ceiling openings. The capacity of a lift depends on its engine power and the size of its container. Typically, they have a capacity of up to 4 cubic meters. Figure 3.86 illustrates the use of a lift for soil removal at a top-down construction site.

3.2.3.3.9 Innovative Methods

Recent advancements in technology, particularly the development of suction or vacuum excavators, have greatly enhanced soil removal operations in top-down construction. These machines can

operate at depths of up to 50 meters, using a suction mechanism to transfer soil through a pipe with a diameter of 15 to 25 centimeters, depending on the machine's power. Their containers can hold between 1 and 10 cubic meters of soil. By replacing multiple pieces of equipment, suction excavators streamline the excavation process and reduce labor requirements. Figures 3.87 and 3.88 illustrate a schematic view of a suction excavator and an example of its application, respectively.



Figure 3.86 A lift transferring soil at a top-down construction site

3.2.3.4 Optimal Planning for Underground Excavation Operations

Excavation and soil removal is an important part of top-down construction and on the critical path of project management. Engineers use various methods to optimize these operations, such as implementing multiple floor slab openings, optimizing opening placements, increasing opening sizes, simultaneous execution of multiple operations, minimizing construction-induced delays, and using optimized excavation procedures.

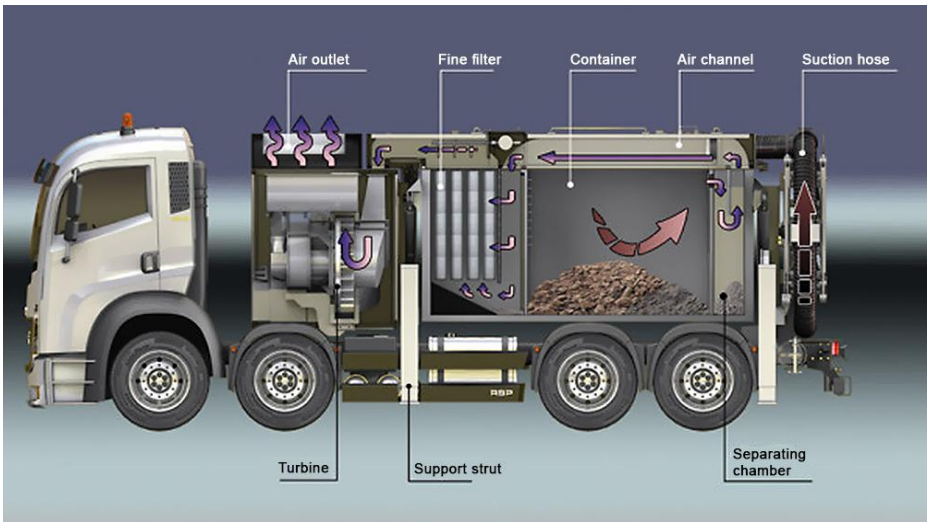


Figure 3.87 A schematic view of a suction excavator and its components



Figure 3.88 A suction excavator

3.2.3.4.1 Optimizing Opening Placements and Sizes

The location, size, and number of openings are determined by various factors. Their placement on the plan depends on the lot's layout, access routes to passages, and the chosen soil removal method. For example, some methods require openings to be positioned adjacent to shaft walls or passages. Additionally, the use of certain equipment, such as tower cranes, may necessitate specific opening placements to optimize the equipment's working efficiency.

The size of an opening is influenced by the adopted soil removal method, the equipment used, and the dimensions of the machinery involved in underground operations, as these machines must be transported to the site through the openings.

If it proves too challenging to meet all operational requirements with a single opening, multiple openings may be necessary. However, this can compromise slab rigidity, and if the openings are located near peripheral retaining walls, their structural stability may also be at risk. Therefore, designing these structures requires extra care and attention to maintain their integrity.

3.2.3.4.2 Wall/Panel Excavation

When planning optimized soil operations, careful consideration must be given to the dimensions, locations, and methods of panel excavation. Larger panel dimensions and uninterrupted construction can expedite excavation; however, they may compromise shaft safety. Geotechnical conditions at the construction site should guide the selection of panel sizes and excavation speed. While efficiency is important, safety must always take precedence.

To mitigate excavation-induced deformations, it may be necessary to retain soil berms adjacent to walls or adopt an alternating panel excavation sequence. Geotechnical experts should specify panel sizes and excavation sequencing based on soil stability, with adjustments made as needed throughout the project to reflect variations in the soil profile.

To strike a balance between speed and safety, temporary safety measures such as steel reinforcements or shotcrete may be required.

3.2.3.4.3 Optimal Operational Planning for Excavation

Operational planning involves executing procedures in a manner that allows for the simultaneous and parallel completion of multiple tasks, minimizes operational conflicts, maximizes excavation stability and safety, and ensures the process is completed as efficiently as possible.

An example of excavation optimization, as proposed by Li et al. (2014) for the top-down construction of a metro station in Shanghai, is illustrated in Figure 3.89.

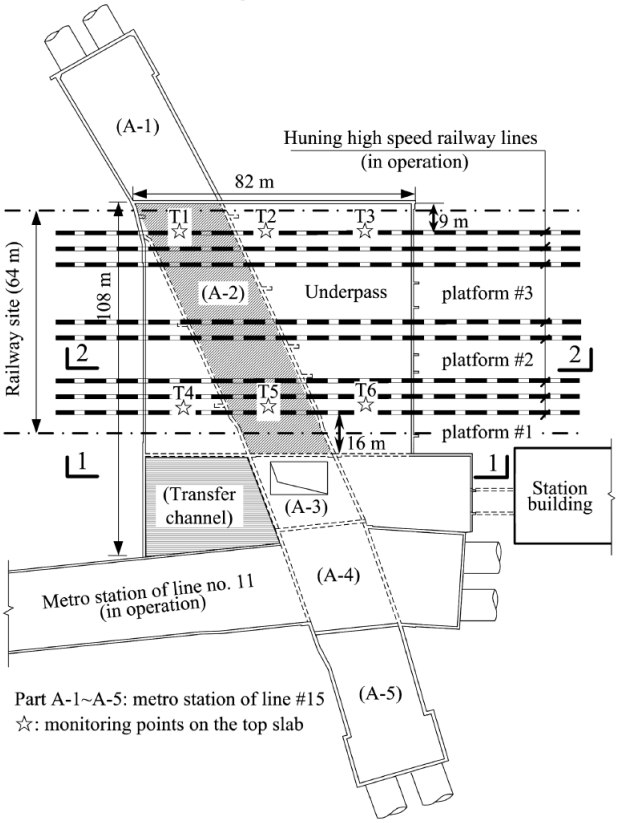


Figure 3.89 A site plan of railway lines and metro stations for a top-down project in Shanghai

In this project, several excavation methods were evaluated, including the conventional top-down method, divided excavation, entire excavation, and segmented excavation. Ultimately, the channel type excavation method was chosen. The following sections provide a detailed discussion of each method.

3.2.3.4.3.1 Divided Excavation via Overhead Openings

In this method, as shown in Figure 3.90a, two or three openings are created in the concrete floor slab of the A-2 section, which serves as the underground ceiling of the underpass. Excavation operations are divided into smaller, independent portions, with work proceeding simultaneously through each opening. Excavated soil is moved vertically through the openings and then transported horizontally to the main exit.

The excavation at the A-2 section is vertically segmented into four layers, aligned with the floor slabs and anchorages. On the south side of the upper floor slab, a small opening is installed to facilitate material delivery. Excavated soil and materials are transported to and stored on the upper floor slab of the A-2 section.

This method was deemed unsuitable for the project due to the limited height clearance of the underpass, which is only 4.5 meters. An excavator equipped with an opening or clamshell bucket could not be used to transfer the excavated soil vertically or move it off-site. Additionally, small excavators could not operate on the floor slab of the A-2 section. Furthermore, since the railway line is in operation, large openings for soil removal could not be installed without compromising the floor slab's load-bearing capacity, thereby increasing project risks.

3.2.3.4.3.2 Entire Excavation and Construction

Given the impossibility of soil transfer through the A-2 section's floor slab, the entire excavation method, as illustrated in Figure 3.90b, was considered. In this approach, the A-2 section is divided into four vertical layers, with soil removed continuously from the south side and transported across the excavated surface to the exit.

Excavating each layer would take over 12 days, and the construction and curing times for floor slabs and anchors would be considerably longer. Prolonged construction timelines, combined with the lack of anchorage for diaphragm walls, would lead to substantial deformations in Shanghai's soft clay soil. Analyses indicate that the deformations caused by the entire excavation method would be far more severe than those associated with the divided excavation method (Zu, 2007).

3.2.3.4.3.3 Segmented Excavation

Segmented excavation, illustrated in Figure 3.90c, is an effective method for controlling diaphragm wall deformations. This approach divides the excavation of each layer into several segments. In each segment, soil is excavated quickly, and anchors and floor slabs are immediately constructed to minimize deformations. Once the concrete reaches 70% of its maximum strength—typically achieved seven days into the curing process—excavation and construction proceed to the next segment.

While this method effectively reduces diaphragm wall deformations, its efficiency is relatively low due to the interruptions caused by the concrete pouring and curing phases. Consequently, construction costs and associated risks are higher.

3.2.3.4.3.4 Channel Type Excavation

In this method, an operational channel for soil transfer is constructed across the excavation site at the start of the project, leaving the remaining soil on both sides of the channel to enhance the diaphragm wall's resistance to deformations. This channel allows excavation to begin at the farthest point from the discharge opening, eliminating the need to pause soil excavation during the construction of the structure and floor slabs.

Once the channel has been fully established, soil on both sides can be excavated using the segmented excavation method. This can take place in parallel with the construction of anchors and floor slabs. Additionally, the channel serves as a pathway for transferring both soil and materials.

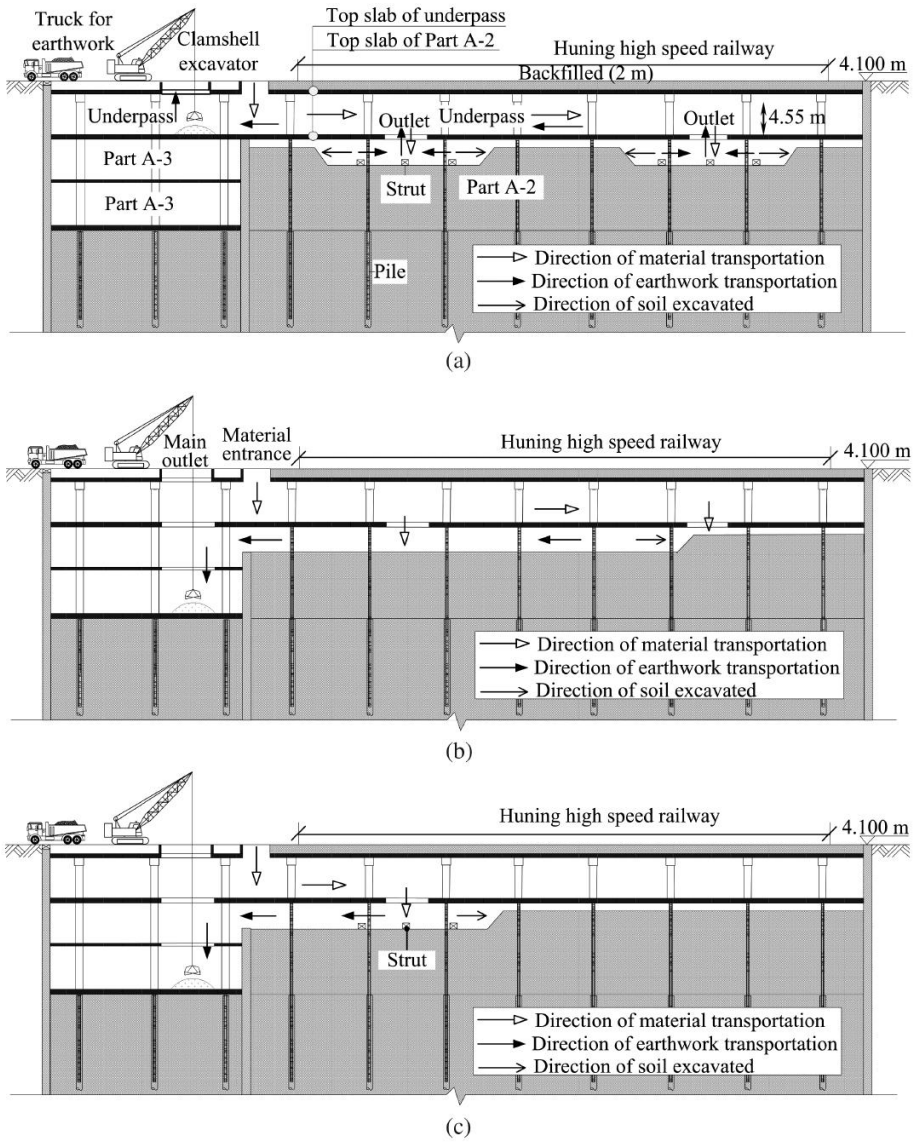


Figure 3.90 a) Divided excavation with overhead openings; b) Entire excavation and construction; c) Segmented excavation

Two factors determine the width of the channel. First, it must be wide enough to accommodate soil transfer and allow excavation machinery to operate effectively. As such, wider channels are generally preferable, but the second consideration is ensuring that sufficient soil remains on both sides of the channel to prevent diaphragm wall deformations. Therefore, the optimal channel width is the minimum necessary to support excavation machinery traffic while maintaining wall stability.

In channel type excavation, construction and excavation progress in opposite directions, ensuring that the two operations do not interfere with each other, while diaphragm wall deformations are also effectively controlled. By minimizing the challenges associated with overlapping excavation and construction activities, the channel type excavation method offers significant improvements. Additionally, compared to other common excavation methods, it saves time and provides enhanced control over excavation-induced deformations.

3.2.4 Construction of Other Structural Components

Top-down construction comprises two primary components: underground and aboveground construction. The aboveground operations in this method are similar to those in conventional construction techniques. Accordingly, the following sections focus on the construction procedures for structural walls, floor slabs, mat foundations, strip foundations, and basement floors.

3.2.4.1 Site Leveling and Preparation for Structural Execution after Installing Vertical Load-Bearing Components

After all vertical load-bearing components have been installed, the project site must be prepared for the next phase of construction. This preparation includes water proofing (if required), constructing the retaining wall (if applicable) using the top-down method, and leveling the site for floor slab construction. Figures 3.91 and 3.92 illustrate a construction site following the installation of columns.



Figure 3.91 Steel column installation and shaft filling, followed by site leveling, to prepare for the next phase of top-down construction



Figure 3.92 Installation of precast concrete columns at the site, following shaft filling, and site preparation for the construction of the floor slab and retaining wall at the first basement level

With project management considerations and execution sequence in mind, some procedures can be performed simultaneously. For instance, once columns on one side of the site are installed and sufficient workspace is available, the next construction phases—including building the floor slab and concrete wall—can begin at a designated point on the site (Figure 3.93). Figure 3.94 shows the preparation for the construction of the retaining wall.



Figure 3.93 Preparation basement before construction operations can begin; defining various work fronts in the project (concrete frame and columns with steel cores at basement levels)

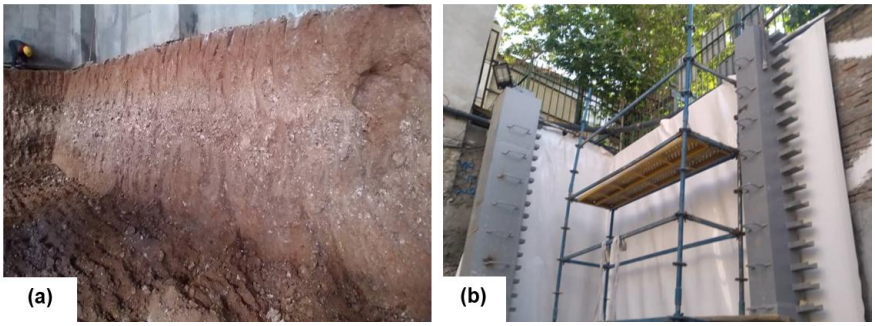


Figure 3.94 a) Panel excavation and leveling of the shaft wall in preparation for the construction of the retaining wall; b) water proofing the excavation wall before the construction of the retaining wall

3.2.4.2 Construction of Retaining and Shear Walls

Once the shaft is filled with a suitable mixture and the site is properly leveled, the structure of the first underground floor can be completed. At this stage, the primary focus should be on constructing the peripheral (retaining) walls and shear walls.

In top-down construction, even when a continuous embedded retaining wall is employed, a concrete wall may still be built over pile walls (Figure 3.95). Diaphragm walls, which serve a structural

function, act as the permanent retaining walls of the structure. These walls are sequentially connected to the floor slab and foundation as soil excavation progresses (Figure 3.96).

In systems where walls are constructed as embedded elements, the execution of basement floors requires careful consideration of construction joints and connections between the walls, floor slabs, and columns. These connections can be established by installing appropriate attachments or rebar planting, following execution guidelines provided by the top-down construction advisor, and in compliance with relevant regulations.



Figure 3.95 Sequential, top-down construction of a retaining wall over an embedded pile retaining wall



Figure 3.96 Connection of a diaphragm, retaining wall to floor slab, and foundation via rebar planting

A construction, or cold joint, forms when concrete pouring for two adjacent components occur at different times. The interface between the freshly poured concrete and the setting concrete is referred to as the pouring surface or cold joint. According to

established regulations, certain considerations must be observed regarding construction joints.

Determining the precise location of cold joints is critical, as they are numerous and unavoidable in top-down construction. Their type and position must be indicated on shop drawings, and identifying their exact location should not be delayed under any circumstances.

When cold joints form, the concrete surface must be thoroughly cleaned, and any laitance and dirt must be carefully removed. Cold joints should be located at points subject to minimal forces, particularly shear forces. If necessary, appropriate measures should be implemented to transfer shear and other forces away from the cold joint.

Ensuring full coherence between the two concrete surfaces at cold joints is essential. To achieve this, when the concrete on one side is still uncured, the top layer of its surface should be removed to expose the aggregates, creating a roughened texture. This can be accomplished by spraying water or a water-air mixture at suitable pressure and using a wire brush.

The treated surface must remain wet until concrete is poured on the adjacent side, except for the cold joint surface itself. Treatment of the joint surface should be stopped several hours before pouring to achieve a shallow dryness. This process is known as green cutting.

Cold joints should not be irregular; they must be formed perpendicular to the axis of vertical stresses. Large cold joints should be avoided whenever possible. If cold joints are necessary, they should be designed in a stepped pattern with broken lines.

Creating proper pouring surfaces requires a temporary formwork. Typically, fine-grid wire mesh, supported by a firm formwork or rabitz, is used. The mesh can either remain embedded in the concrete or can be removed at an appropriate time. This approach

helps create a roughened surface. Before pouring concrete on the second side, the pouring surface must be thoroughly prepared. It should be free of particles, oil, grease, paint, and similar substances. Cleaning should continue until the gravel particles are exposed.

The most effective method for cleaning the surface is spraying wet sand using a water jet. Alternative methods, such as pickling or using manual tools, can also be employed depending on the position of the cold joint. To ensure proper coherence between the fresh and setting concrete, once the pouring surface is broken during the top-down construction process, it should be kept wet for an extended period. Before the next round of pouring, surface water should be removed using compressed air. For improved coherence, a concrete mixture with higher workability is recommended. This can be achieved by increasing the slump, using more gravel, or reducing the number of coarse aggregates. Enhanced coherence can also be achieved by thoroughly vibrating the initial layers of the freshly poured concrete.

For the sequential, top-down construction of retaining walls—a common method in Iran where the soil is naturally stable in panel with limited dimensions between columns spans (a condition prevalent in most district of Tehran alluvium)—several executional considerations must be addressed in addition to those related to cold joints.

After site leveling, if waterproofing of the excavation site's soil wall is required, the retaining wall's reinforcement cage is installed between the structure's two columns.

Regarding the vertical reinforcements of the wall, it is essential to account for rebar extensions and connections when preparing rebar cut lists and shop drawings. This is particularly important because, in the subsequent phase, the upper part of the basement wall will connect to the lower part of the upper basement floor.

The connection of the upper floor rebars to the lower floor ones can be achieved either through overlapping lengths or couplers. In both cases, the length of the rebar connection, which is situated below the floor slab, must be safeguarded to ensure that the lower part of the wall is executed with precision. This allows for the proper connection of the longitudinal rebars for the next basement floor. The placement of couplers should be determined based on the recommendations of the top-down construction expert.

Depending on the executional plan and the required length to secure overlaps or couplers, various methods can be utilized. If the overlap or coupler length is significant, a trench matching the required depth must be excavated at the wall's footing. However, this approach can be costly and time-consuming, especially when trenches deeper than 70 centimeters are required. Given these challenges, such trenches are often excavated manually.

After trench excavation, the wall's vertical rebars are installed, with the portion requiring safeguarding placed within the trench. The trench is then backfilled with soil. To ensure a clean and well-prepared cold joint beneath the wall, the surface of the concrete below the wall and above the soil must be carefully prepared. Typically, a layer of sand is applied to the surface. However, to prevent the sand from mixing with falling concrete, it is advisable to pour a thin layer of cement grout over the sand.

When backfilling the trench, it is recommended to keep the soil moist and compacted using water to minimize settlement. The weight of the freshly poured concrete, combined with its inherent shrinkage, can exacerbate soil settlement beneath the wall, potentially compromising the structure. To counter this, the concrete pouring level should be slightly higher than planned initially, ensuring that after settlement and shrinkage, the final level matches the executional design.

In cases where the splice length beneath the wall is minimal, alternatives such as attaching rabitz or similar materials to the reinforcement cage at the wall's base can be employed to secure

the splice. A sand layer, paired with a thin cement grout coating, can then be used to prepare the wall's bottom surface, as described earlier. Ultimately, any method that adequately addresses construction joint considerations and maintains concrete quality is acceptable for execution.Figure 3.97 illustrates an example of safeguarding the splice length of longitudinal rebars at the base of a retaining wall. When couplers are used to connect upper rebars to lower ones, it is crucial to adequately cover the connection to protect it from concrete and other construction materials during the retaining wall's construction. Figure 3.98 depicts a similar scenario where the entire coupler length is covered with plastic film. After concrete pouring is completed, the plastic film is removed, allowing the rebar to be connected to its corresponding floor.

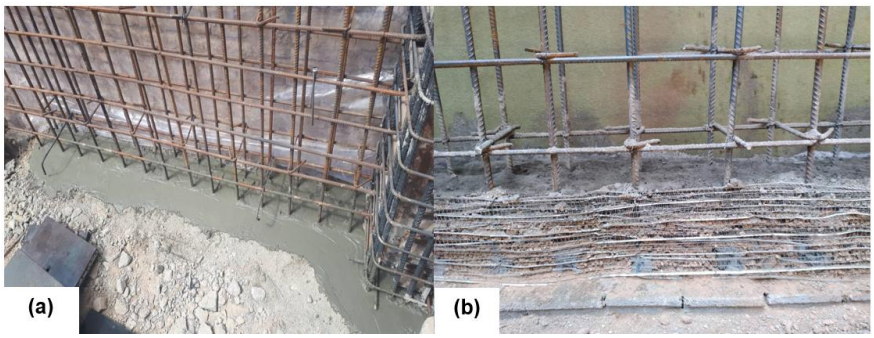


Figure 3.97 Protecting the starter bar length at the base of retaining walls where longitudinal rebars of each level are connected: a) Trench excavation at the base of the wall. Note the positioning of the primary rebars, and backfilling the trench with a gravel cover and a thin layer of cement grout; b) Protecting rebars below the level of the retaining wall's base using rabitz and backfilling the length of the splice with soil, gravel, and a thin layer of cement grout

For transversal rebars in walls, standard construction methods can typically be used. In the case of steel columns or steel cores within composite concrete columns, primary reinforcement is either connected to or positioned adjacent to the steel column or core, as

per design. Overlapping lengths or couplers can then be used for extensions and connections. Since transversal reinforcement is generally thinner, overlapping lengths are both more practical and more commonly employed.



Figure 3.98 Protecting couplers with plastic film covers to prevent splice blocks and groove damage

An important consideration for transversal reinforcement is its connection to precast concrete columns. If the entire precast column cannot be embedded within the retaining wall, and transverse rebars cannot pass through both sides of the column, these rebars must be anchored to the column. In such cases, as with both precast and cast-in-place concrete columns, starter bar as mechanical couplers, are pre-installed in the column as specified in shop drawings. Once soil excavation is complete, transversal rebars are placed between the columns (Figure 3.99). Figure 3.100 depicts a retaining wall prepared for the formwork.



Figure 3.99 Connecting retaining wall's transverse rebars to a precast concrete column through starter bars as mechanical couplers installed during column construction



Figure 3.100 Reinforcement cage on a retaining wall prepared for formwork and concrete pouring

After the reinforcement cage is installed and all preparations are complete, the next step is to install the formwork. Since the formwork for retaining walls is executed using the one-sided

method, anchoring the formwork is a critical consideration. Several techniques can be employed for installing retaining wall formwork. The simplest and most commonly used approach is to utilize the ground as a support for wall braces (Figure 3.101). To ensure the ground can withstand the significant pressure of concrete, a suitable support position must be identified. This can be achieved by excavating a shallow trench and constructing an embedded horizontal footing or by installing concrete or steel bases securely anchored to the ground. However, creating reliable support on the ground can present execution challenges. Additionally, using braces at the corners of the wall can result in interference between them, potentially complicating the formwork installation process. Careful planning and execution are therefore essential to ensure stability and efficiency.



Figure 3.101 Anchoring one-sided formwork to supports on the ground via rams

Another method for installing a one-sided formwork on a retaining wall involves using the installed structural columns as supports, as illustrated in Figure 3.102. Alternatively, a chassis can be installed to use the central columns as support, as shown in Figure 3.103.

Managing the significant pressure exerted by concrete is a critical consideration during retaining wall construction. It is advisable to select an appropriate section and anchoring method based on precise formwork calculations. Figure 3.104 provides another

example of one-sided anchorage for a retaining wall using steel truss supports.



Figure 3.102 Steel support installation and anchorage of a one-sided formwork using structural columns



Figure 3.103 One-sided anchorage of a retaining wall to central columns via a chassis

In retaining wall sections that extend from the foundation to the upper floors without being connected to a floor slab—such as elevator shaft walls or other vertical openings constructed sequentially from top to bottom—or in the lower parts of walls formed with cold joints, concrete is typically poured through an angled hatch placed at the top of the wall. To account for potential concrete shrinkage and settlement on ground soil, the pour is

made slightly above the base of the upper wall segment. After the initial setting, any excess concrete can be trimmed off (Figure 3.105). Alternative, more innovative pouring methods may also be considered in such cases.



Figure 3.104 Retaining wall anchorage using truss supports

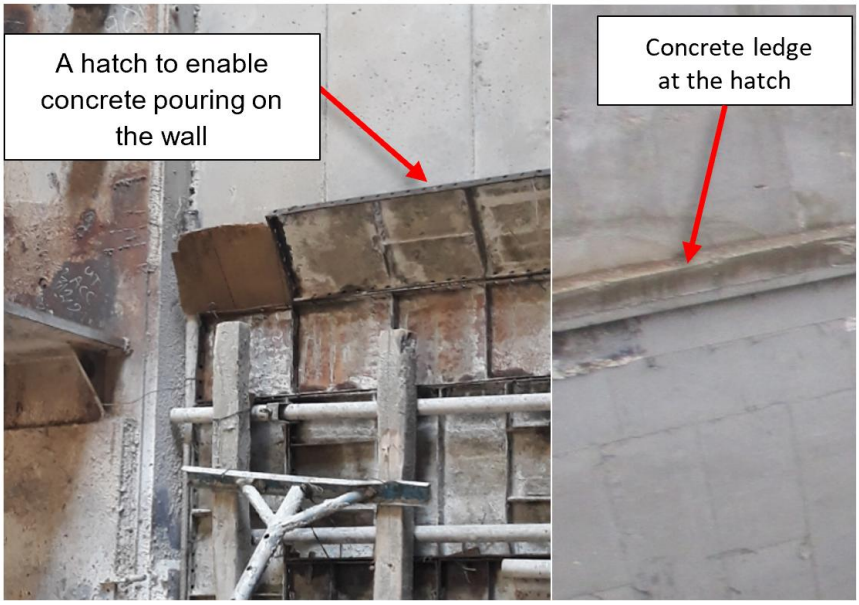


Figure 3.105 A concrete pouring hatch at the top of a wall; and the excess concrete that should be cleared away

Similar to retaining walls, shear walls can be built sequentially from top to bottom according to top-down design principles. The installation of formwork for shear walls follows the same guidelines as retaining walls, and, despite the use of two-sided formworks, no additional specialized measures are required for top-down construction. Figure 3.106 illustrates a two-sided formwork on a shear wall constructed sequentially from top to bottom.

Figure 3.107 depicts the top-down construction of a wall incorporating a steel core. This steel core is accounted for during the modeling phase of top-down construction analysis and design, as well as in the preparation of shop drawings. Like steel cores in composite concrete columns, this load-bearing component transfers loads to a concrete pile during the top-down construction process.



Figure 3.106 Two-sided formwork and top-down construction of a shear wall



Figure 3.107 Top-down construction of a shear wall where the vertical load-bearing component is used to transfer loads to the concrete pile

3.2.4.3 Floor Slab Execution

Various floor slab systems can be utilized in top-down construction, including steel deck, composite, waffle, ribbed, and beam-and-block slabs. The selection of an appropriate slab system for basement floors should be guided by the structural engineer's recommendations regarding load requirements and the top-down construction engineer's insights regarding executional considerations.

Concrete slabs require sufficient curing time to achieve at least 70% of their nominal design strength before formwork can be removed from them. This curing process can delay underground excavation activities, as commencing these operations before the removal of formwork is generally impractical.

For slabs incorporating steel beams, challenges can arise when transferring and installing beams after concrete pouring, as crane access may be restricted. However, in these cases, excavation can proceed immediately following concrete casting. The transfer and installation of steel beams post-pouring can be facilitated using equipment such as mini loaders or mini excavators.

In some instances, such as with steel deck slabs where steel beams are installed first, top-down construction principles and proper design allow the beams to remain self-supporting until excavation reaches the ground level of the shaft. Once excavation is complete, the floor slab can be finalized by pouring concrete. It is important to note that in this method, beam dimensions must be larger compared to scenarios where the beams, in conjunction with the slab's diaphragm, serve as retaining wall anchors to counter soil loads. This adjustment can offer significant architectural and cost-saving advantages. Figure 3.108 illustrates a steel deck floor slab at a top-down construction site.

Concrete floor slab formworks can be constructed using various methods. Formworks may be supported by rams and scaffolding or directly by the ground surface. In either case, the soil must be properly leveled and compacted if necessary.

When rams are used to support the formwork beneath the floor slab, it is essential to provide adequate ground support beneath each ram. This ensures that the loads from the poured floor slab and the formwork equipment are safely transferred to the ground without risk of collapse or settlement.

Figure 3.109 illustrates a floor slab formwork for a waffle slab, where rams have been utilized. Figure 3.110 shows the use of wooden timbers to transfer loads from the floor slab to the ground. In such cases, it is critical to level the ground before placing the wooden timbers. In remolded or soft soil conditions, the soil must be compacted using suitable equipment, tools, and methods to ensure stability.

If properly prepared, the ground surface can serve as formwork for a floor slab, eliminating the need for rams. To achieve this, the surface must first be leveled. A layer of plywood or lean concrete can then be laid, followed by the installation of a plastic sheet to serve as a base for pouring the slab's concrete (Figure 3.111).



Figure 3.108 Steel deck floor slab at a top-down construction site

For both precast and cast-in-place concrete columns, all required connections must be pre-installed at the slab level within the column. Figure 3.112 illustrates a floor slab with beam connections to a precast concrete column.

To expedite the removal of the formwork and allow excavation operations to proceed below the floor slab level, the concrete mixture for the slab can be specifically designed to achieve the required strength more quickly.

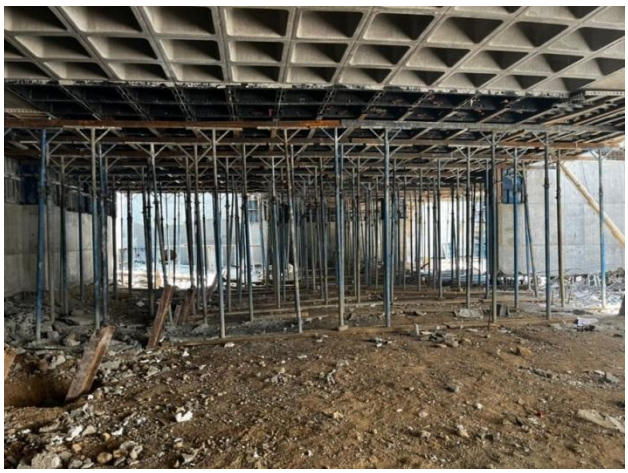


Figure 3.109 Formwork installation on rams established on the soil surface under a waffle slab



Figure 3.110 Wood timbers under rams to transfer the load from the recently cast floor slab and formwork equipment to the ground, preventing ram settlement into the soil as the floor slabs are under construction



Figure 3.111 Beam-and-block floor slabs at a construction site: a) using plywood for formwork; b) floor slab on a plastic film layer



Figure 3.112 Beam-and-block floor slabs with beam connections to precast concrete columns at a top-down construction site

3.2.4.4 Execution of Foundations

The execution of the foundation marks one of the final stages of underground construction in the top-down method. Similar to bottom-up construction, the process begins by leveling the soil and casting a layer of lean concrete. Subsequently, reinforcement is installed following the same approach as in the bottom-up method. A critical consideration at this stage is the anchorage of pile rebars within the foundation. If the design specifies this requirement and it is outlined in the shop drawings, the pile rebars must be securely anchored into the foundation (Figure 3.113).

For both precast and cast-in-place concrete columns, waiting connection rebars for linking the column to the foundation must be pre-installed. This ensures that when the foundation's reinforcement cage is being installed, the connection between the concrete column and the foundation can be seamlessly established using these pre-installed rebars (Figure 3.114).

Steel columns installed using the SPW approach require a base plate to be placed on the foundation, facilitating the anchorage of the column to the foundation. This process is guided by the construction engineer's recommendations informed by structural design considerations. Once the foundation reinforcement is complete, concrete can be poured.



Figure 3.113 Reinforcement installation and concrete pouring at a top-down construction site. Note the steel columns and the pile starter bars.



Figure 3.114 Execution of the foundation. Note the precast concrete columns with the preinstalled starter bars.



Figure 3.115 A steel column's two-piece base plate at the foundation level

3.2.4.5 Finalizing Underground Construction and Closing Temporary Openings

Once the foundation concrete is poured and all pieces of equipment, including excavators, are removed from the site, any temporary openings—such as those used for soil removal or material intake during top-down construction—should be sealed from the bottom up. This process finalizes basement structures, completing the underground construction phase.

3.3 Critical Considerations in Top-Down Construction

When using the top-down method, construction and excavation operations proceed concurrently. As a result, this approach necessitates attention to specific critical factors. The most significant of these considerations are detailed in the following sections.

3.3.1 Superstructure Considerations

As the name suggests, the top-down method enables simultaneous construction above and below ground level, providing a significant advantage in project time management. This approach also supports the phased operation of the superstructure as outlined in the second section of this guideline. Some experts argue that the top-down method is only fully implemented when phased operation is achieved. When the superstructure remains inoperable during construction, they refer to the method as 'semi-top-down.'

Leveraging this advantage requires meticulous structural design and stringent structural controls to address the demands of an operational superstructure and its loads. Constructing aboveground levels before completing the mat or strip foundation necessitates that the substructure and initial foundation (used during the construction phase) are designed to safely bear the additional loads from the superstructure.

To this end, during the structural design control phase and the analysis and design of a top-down project, it is essential to account for the loads from the aboveground levels. The number of superstructure levels to be constructed concurrently with excavation and underground construction, as well as the planned operational timeline during construction, are critical considerations when designing construction procedures.

Before initiating structural analysis and design, it is imperative to establish a detailed execution schedule and construction timeline for aboveground levels. This requires collaboration between construction teams, project managers, clients, and other stakeholders.

3.3.2 Permissible Tolerances in Structure Construction

Currently, there are no dedicated national or international codes and regulations specific to the top-down construction method.

However, tolerances for individual operational phases are often defined and should be referenced by constructors. For instance, column installation tolerances—including misalignment and eccentricity—are outlined in Chapters 9, 10, and 11 of the Iranian National Building Codes. Additionally, the latest edition of the *General Building Technical Specifications* (Publication 55) includes chapters that address aspects of the top-down method.

It is important to note that many of the tolerances mentioned in these references are designed for conventional bottom-to-top construction. Adapting them to the unique requirements of top-down construction can present challenges. The development and implementation of tolerances tailored to the top-down method are recommended.

3.3.3 Groundwater Challenges

Groundwater can be encountered in two distinct stages of top-down construction: during the excavation phase and during the operational phase. Managing groundwater in these situations requires careful attention to specific considerations. The table below categorizes these considerations into two groups: temporary measures for the excavation phase and permanent measures for the operational phase.
Groundwater management during the operational phase of the top-down method is not fundamentally different from other construction methods. However, when employing the SPW approach without diaphragm walls, waterproofing and drainage can be more complex due to factors such as underground operations, constrained work front, and the presence of numerous cold joints in concrete.

The most critical consideration for permanent waterproofing in top-down projects is the progressive establishment of manageable work fronts. This approach necessitates insulating small, segmented portions, which increases the number of insulation gaps. Additionally, as walls are constructed in multiple stages, numerous cold joints will inevitably. To mitigate the risks

associated with cold joints, adherence to codes and regulations related to concrete cold joints, as outlined in this guidebook, is essential.

Table 3.1 Groundwater management measures

Excavation Phase	Lowering the groundwater level using wells and dewatering techniques
	Grouting to create a waterproof barrier on the floor
	Concrete pouring in water-affected areas
Operational Phase	Permanent groundwater table lowering through drainage systems
	Waterproof flooring through floor anchoring
	Thick, watertight flooring through weight-based impermeability

As the number of gaps and cold joints increases, the risk of seepage goes up significantly. Specific joints, such as those between concrete and steel columns—which are common in the top-down method—might call for special measures. Effective water seals and waterstops, such as hydrophilic or bentonite-based solutions, are required to ensure a robust waterproofing system.

In the conventional top-down method, where a concrete wall encloses the construction site, groundwater management is a concern from the outset. Since groundwater control measures are implemented before excavation begins, this approach is often considered superior to others. However, a critical challenge in this approach is ensuring proper waterproofing at the joints between different sections of the concrete walls, which must be addressed with meticulous attention.

In contrast, the SPW approach to the top-down method—where the peripheral wall is constructed in stages—presents a different set of challenges. As excavation begins without prior preventive measures, groundwater issues arise progressively during the project, leading to more significant difficulties in managing groundwater effectively.

Typically, engineers have three options to address these challenges:

a) Dewatering to remove water from the site
b) Drainage or guiding waters at the project site through designated routes
c) Waterproofing or preventing water from reaching the project site

Water drainage and waterproofing in underground areas are achieved by adhering to the codes and regulations outlined in Publication 55 and Chapter 7 of the Iranian National Building Codes. Additionally, established international standards, such as BS 8102:2009 (ICS DIN 18195 and 91.120.30), should be consulted. British standards provide three types of protection (A, B, and C) designed for underground areas. Depending on the chosen protection type, various dewatering, drainage, and waterproofing methods can be implemented, including the installation of impermeable membranes and the use of waterproof concrete, under the standards mentioned above

3.3.3.1 Water Sources

Water encountered during excavation at top-down construction sites can originate from various sources. Identifying these sources is crucial for engineers to develop effective water management strategies. The water may be present because the excavation site is located below the natural groundwater table, it might stem from a local underground water source or channel, or it could come from surface water seeping into the excavation area.

Encountering a natural groundwater table poses the most significant challenge, often impacting the entire construction project. The extent of water discharge depends on the soil's permeability, with highly permeable soils presenting particularly difficult conditions. Addressing this issue may require a combination of strategies, including waterproofing and drainage. Common measures include constructing a peripheral waterproof wall, reducing soil permeability through injections, lowering the groundwater table via pumping, and implementing drainage systems both within and around the excavation site.

Local water sources may include sewage systems, pipe ruptures caused by excavation-induced deformations, and enclosed water reservoirs. Such situations are relatively common and generally manageable. However, the key concern is effective management to prevent geotechnical instabilities and hygiene issues. Solutions include creating drainage channels to collect water from the excavation site floor, installing drainage strips behind walls, and excavating horizontal drainage systems.

When qanats are the source of water, the issue is typically localized. However, depending on the qanat's discharge flow, the entire project could be at risk. The optimal approach in such cases is to redirect the water flow without obstruction, ensuring it does not enter the project area while minimizing disruption to construction operations.

When water originates from a surface source and enters the excavation site through soil seepage or channels, the most effective approach is to prevent its entry into the site by redirecting it at a safe distance away from the excavation walls.

3.3.3.2 Dewatering

The most secure approach to managing water is to divert it before it reaches the project site. This method involves drainage outside

the excavation shaft and lowering the water table to at least the level of the excavation shaft bottom, effectively eliminating the risk of water entering the shaft. In the top-down construction method, where excavation equipment operates underground, water intrusion into the shaft can cause significant challenges, including soil becoming waterlogged and muddy. Dewatering is typically employed during the construction phase and is a cost-effective solution for excavation sites with moderate soil permeability. However, during the operational phase, dewatering can be prohibitively expensive and is often impractical.

Various methods can be employed for water drainage outside the excavation shaft, including drilling deep wells (Figure 3.116a), installing well points (Figures 3.116b and 3.116c), constructing horizontal drainage galleries, or using a combination of wells and galleries.

When implementing this approach, water seepage may occur over a wide area surrounding the project site. While some seepage is expected when draining water from excavation sites, using drainage methods outside the excavation shaft can significantly expand the affected zone. It is essential to consult geotechnical experts and hydrologists before adopting such measures.

According to the technical principles of dewatering, regardless of the method used, three essential requirements must be met to lower the groundwater level effectively: 1) fine soil aggregates must remain stable and undisturbed; 2) the shaft floor must remain dry; and 3) the shaft floor and slopes must maintain their stability. To achieve these objectives, wells (as shown in the figures 3.116) are drilled, allowing the groundwater level to gradually recede and enabling excavation operations to proceed.

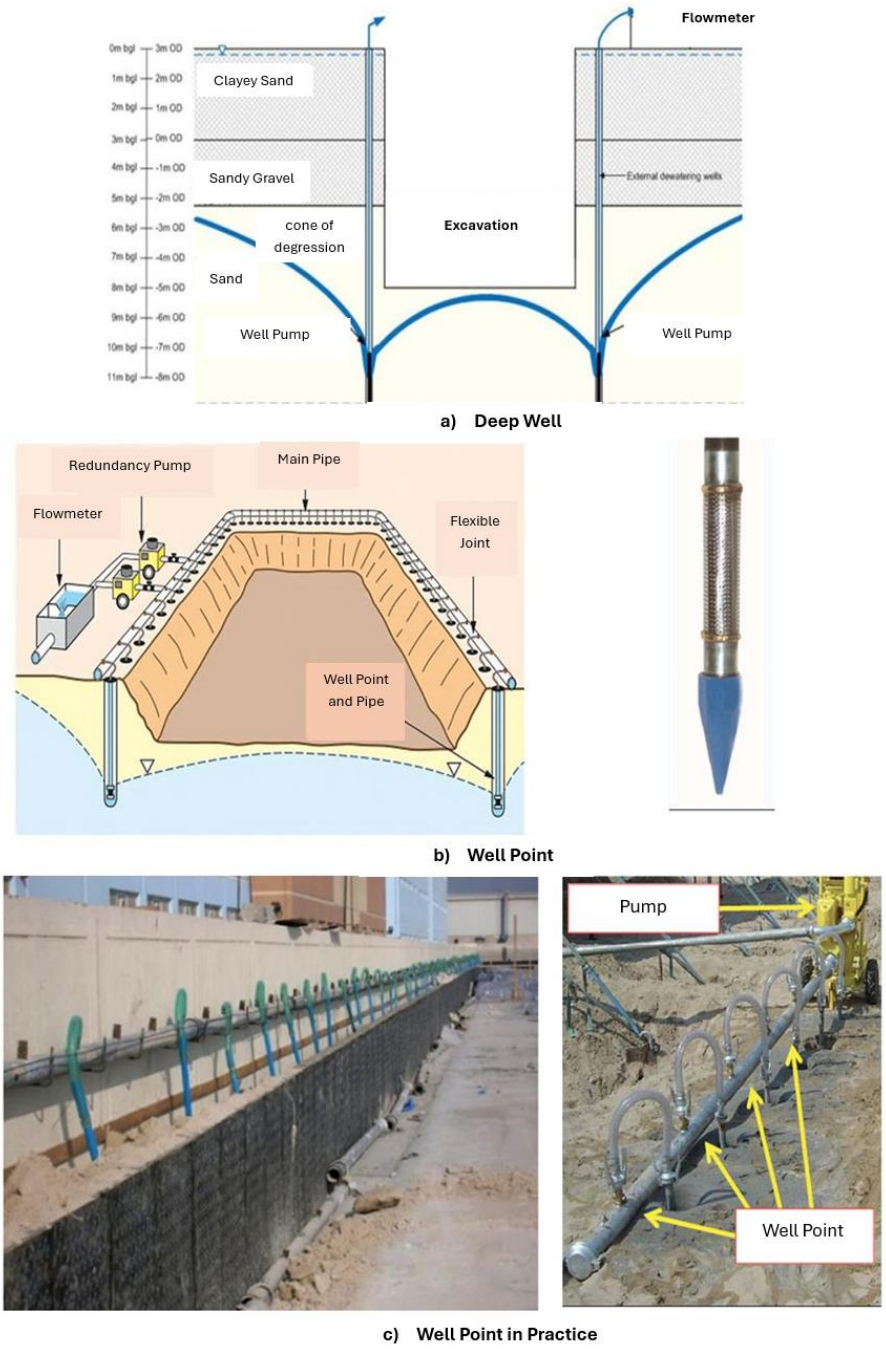


Figure 3.116 Dewatering th e area surrounding a shaft

Well points are pipes with a diameter of 5 centimeters, featuring perforations along the final one meter to facilitate water extraction. As shown in the figures below, each pump can accommodate 25 to 50 pipes. In sandy ground conditions, wells can be drilled, and pipes installed using high-pressure water. This method of dewatering is suitable for soils with low to medium permeability (fine, silty sand) or medium permeability (15^{-4} to 15^{-6} m/s). The typical efficiency of a single well point is approximately 0.5 cubic meters per hour, with potential performance reaching up to 1 cubic meter per hour.

Large wells are installed through drilling, with perforated pipes placed inside. In some soil conditions, these pipes may be ineffective. The area around the perforated section of the pipe is backfilled with sand and gravel, creating a filter that prevents soil particles from entering the pipe despite the suction force from the pump. These wells can range in diameter from 0.2 meters to several meters. Temporary wells typically have a diameter of 0.3 to 0.4 meters. For permanent wells, plastic pipes are used to eliminate the risk of corrosion.

3.3.3.3 Drainage

If dewatering is not adequate, water can reach the shaft wall and seep into the excavation shaft. Any water within or behind the shaft wall must be managed effectively, regardless of its source—groundwater, local water, or a qanat. To achieve this, ditches are created within the shaft area, allowing the infiltrated water to flow by gravity toward a sump. From the sump, the water is then pumped out to a designated area outside the shaft (Figure 3.117).

Determining the discharge rate and selecting a suitable pump for this method depend on various factors, including soil permeability, and require thorough geotechnical analysis. While water drainage and removal from the shaft differ between the construction and operational phases, certain principles are shared

between them. In both phases, water that reaches the shaft wall is redirected through designated channels and subsequently pumped out of the shaft.

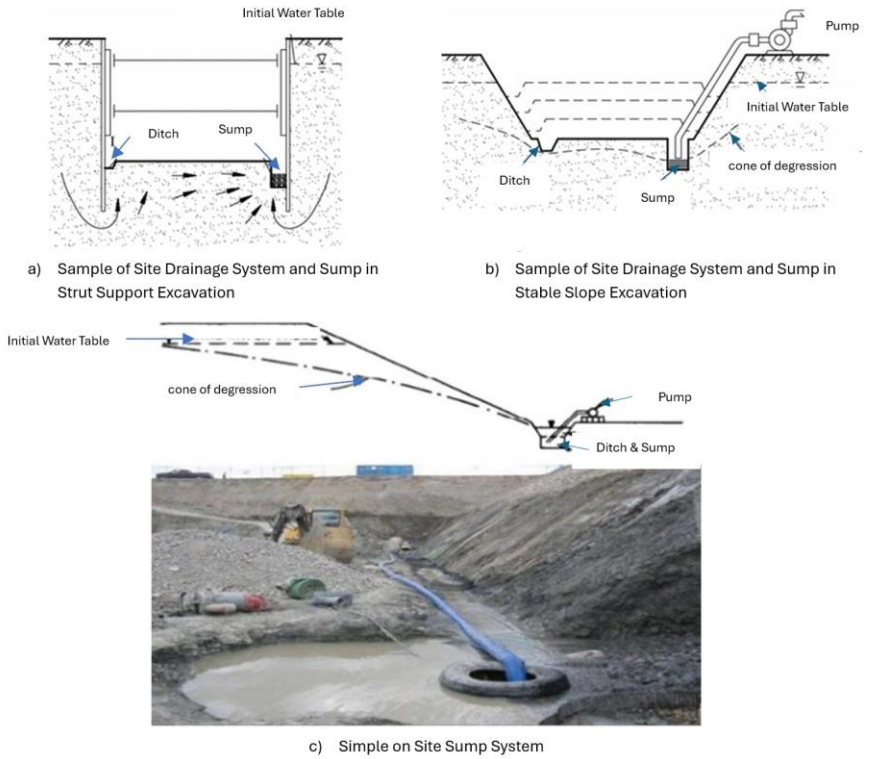


Figure 3.117 Drainage from a shaft

During the construction phase, drainage can be managed by installing strip or depth drainage systems within the shaft wall to absorb water reaching the wall. The collected drain water is channeled through designated pathways and redirected via ditches toward a sump. From there, it is pumped out of the shaft. These measures effectively control and remove water from within the shaft, ensuring safety throughout the construction process.

The same process applies during the operational phase. Strip drainage systems or dimpled sheets are used to collect water reaching the walls and direct it to the structure's lowest point

(Figures 3.118a and 3.118b). At this point, the collected water enters perforated pipes (Figure 3.118e) and is filtered through an appropriate system (Figure 3.118d) enclosed within a porous, sandy environment (Figure 3.118a). The layout of perforated pipes can follow various patterns, some of which are illustrated in Figure 1.119. Figure 1.120 shows an example of a sump and its detailed components.

Drainage systems are generally classified into two main categories: surface systems, primarily used during the construction phase, and subsurface systems, which are predominantly utilized during the operational phase. Subsurface systems are further divided into two types: fin drains and filter drains.

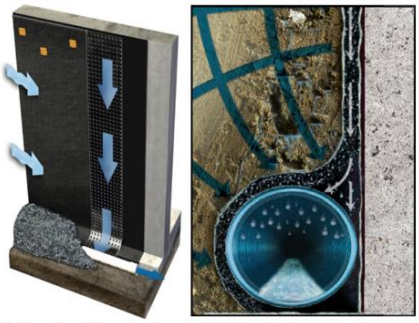
Drains can be categorized based on their material into two main groups: those made from natural materials and those made from artificial materials. Furthermore, they can be categorized into strip, trench, triangular, multi-faceted, and circular, based on their shape and cross-section.

To ensure optimal efficiency and ease of construction, surface drains are typically designed with trapezoidal cross-sections, following hydraulic principles applicable to open channels.

Subsurface drains are installed below ground and typically operate without direct exposure to air pressure. These drains are categorized into two types: fin drains and filter drains. Their functionality relies on the movement of fluid through the soil from a high-pressure environment to a low-pressure one, making the ability to create a pressure differential a critical factor in selecting suitable drain materials. These materials must possess adequate porosity. Additionally, measures to prevent pore blockage due to sedimentation should be integrated into the system, and the drains should be designed for easy maintenance and repair. Geosynthetic materials are commonly used as they fulfill these requirements effectively.



a) PVC drainage pipes



b) Geotexile-equipped PVC drainage pipes



c) Blockage caused by the absence of an appropriate filter in the drainage pipe



d) Membrane drainage pipes



e) An example of drainage pipes equipped with filters and geodrains



f) An example of the herringbone system at a project site located in Tehran

Figure 3.118 Details of drainage from within the shaft

When designing drainage systems, two critical factors must be addressed: 1) Ensuring the unobstructed flow of water through the system; and 2) Preventing the movement of soil particles along with the water. To achieve these objectives, filters are typically

integrated into the design to minimize the risk of pore blockage and to retain the fine aggregates of the soil, preventing them from entering the drain. Figure 118.c shows an example where the lack of a suitable filter has led to blockage.

One of the most effective drainage methods involves the use of perforated pipes, typically made of PVC. These pipes channel water into their interior through pores or slots in their walls or gaps between adjacent sections. Underground drainage pipes can be installed at various inclinations, a critical factor that influences both the drainage rate and the longevity of the drainage network. Porous pipes are generally embedded in a sand layer, which absorbs surrounding water. To prevent sedimentation and blockage of the pipes, the sand layer is typically covered with geotextile sheets, as illustrated in Figure 1.118d.

Another material commonly used as a fin drain in pipe walls is geodrain. In situations where water seeps from trench walls or excavation shafts, it is essential to channel the water from the wall surfaces to the shaft floor, where it can be collected and drained using pipes installed beneath the foundation. Geodrains, typically made of polyethylene, are among the most widely used materials for this purpose. Figure 3.118b illustrates their function schematically. Porous polyethylene sheets, commonly referred to as dimple sheets, are the most prevalent type.

Geodrain covers are widely used fin drain materials due to their cost-effectiveness, ability to mitigate sedimentation risks, support for mechanically high-resistance materials to protect the waterproof fins, excellent structural protection due to high-discharge capacity, compatibility with embankment materials behind the fin, and their lightweight design and ease of installation. These systems can be easily prepared on the surface and installed quickly after excavation. Additionally, their polyethylene composition ensures durability and long-term performance.

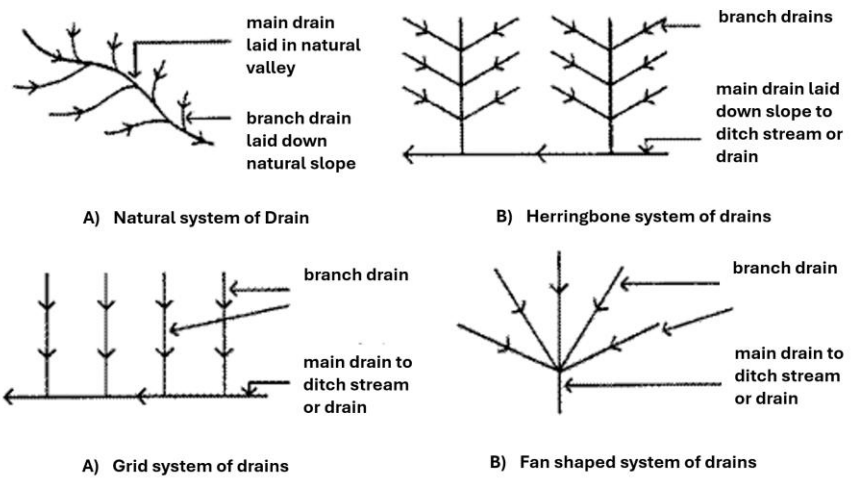


Figure 3.119 Drainage system types

Drains can be configured either parallel or perpendicular to water flow, depending on the topography and the specific type of surface or subsurface drainage system. Figure 3.119 illustrates four common drainage configurations: natural, herringbone, grid, and fan-shaped.

The natural system relies on the principle that water naturally follows exit routes over time based on the site's topography, with drainage systems implemented only at critical points along these routes.

The herringbone system features auxiliary drainage routes arranged at specific angles on either side of a main route. This configuration is particularly well-suited for sloping sites and is often combined with other drainage systems to achieve optimal performance on larger sites with irregular topography.

The grid system comprises auxiliary routes arranged perpendicular to a main route. This configuration is typically employed alongside other systems at sites with regularly shaped lots. With fewer converging points required, this method is simpler to install and maintain compared to many other drainage configurations. However, silt formation is common at these points;

therefore, 90-degree connections should be avoided as much as possible.

The fan-shaped system incorporates auxiliary routes converging on main drainage points. It can be used at big or small sites that have been divided into smaller lots beforehand.

Not all drainage systems adhere to regular configurations. Irregular configurations may be necessary for sites with uneven topography, where wet areas are isolated. However, even in such setups, the main route is typically positioned at the lowest points, with auxiliary routes converging toward it.

Regardless of the drainage system configuration, the route lengths, depths, and capacities must be meticulously planned to align with the overall drainage design requirements of the project.

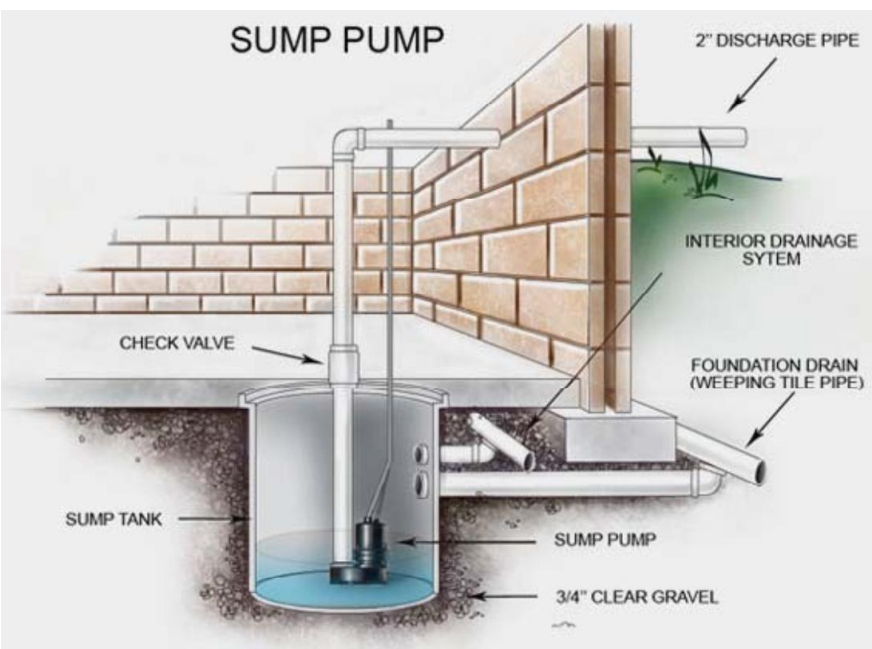


Figure 3.120 Pumping sump details



According to Publication 55, for trenches excavated for drainage, the trench width should range between 35 and 45 centimeters, and the pipes should be surrounded and protected by materials with proper drainage capabilities (filters). Additionally, the particle gradation of filter materials must comply with the following equations:

$$\frac{D_{15,filter}}{D_{85,trench\ soil}} < 5 \quad (3\text{-}1)$$

$$12 < \frac{D_{15,filter}}{D_{15,trench\ soil}} < 40 \quad (3\text{-}2)$$

$$12 < \frac{D_{50,filter}}{D_{50,trench\ soil}} < 58 \quad (3\text{-}3)$$

Where D represents the diameter for particles finer than a percent of particles.

Furthermore, filter materials should meet the following criteria: 100% of the materials must pass through a 37.5-millimeter sieve, 90% must pass through a 19-millimeter sieve, and no more than 10% should pass through a 60-millimeter sieve. Filter materials must be thoroughly washed and free of particles passing through a No. 200 sieve. In areas where slotted drains are used, the D_{85} of the filter material must be greater than half the slot width.

If geosynthetic materials are used as filters, the geotextile must remain intact, allowing water to pass through while effectively retaining soil particles. In other words, the geotextile must simultaneously permit water flow (requiring open pores) and prevent soil particle movement (requiring closed pores). Non-woven geotextiles can be used as filters in drainage systems. Non-woven geotextiles are commonly used as filters in drainage systems. Their interwoven fibers, which distribute loads across multiple threads, allow non-woven geotextiles to be more effective at retaining fine soil aggregates and exhibit better tear resistance

compared to woven geotextiles. The minimum requirements for non-woven geotextiles are the following:

- Tensile Strength: 90 pounds (ASTM D4632)
- Burst Strength: 180 psi (ASTM D3786)
- Post-Rupture Elongation: <50% (ASTM D4833)
- Puncture Resistance: 40 pounds (ASTM D4833)
- Apparent Opening Size (AOS): Up to sieve No. 40 (ASTM D4751)
- Permeability: 0.7 (ASTM D4491)

There are specific recommendations for designing drainage trenches. Ideally, drainage trenches should be 50 to 100 millimeters wider than the pipe diameter. The trench excavation method also plays a crucial role, as damage to trench walls during excavation can obstruct or disrupt water flow. As mentioned above, after installing pipes a layer of filter material is required. Similarly, a layer of filter material, with a minimum thickness of 25 millimeters, should be placed on the trench floor to facilitate smooth water flow. Due to water's tensile forces, it cannot pass through small openings between soil particles to enter pipe pores, making a layer of gravel essential. When sandy materials are used for drainage in the trench, an additional sand layer is necessary to prevent soil particles from infiltrating the drainage materials.

The minimum slope angle for installing drainage pipes is 0.5% (1:200). Installing pipes at slopes lower than this can lead to significant issues, while steeper angles generally improve performance. Additionally, the slope of the main routes should exceed that of the auxiliary routes, and the gradient must be consistent throughout each route. Reducing the slope angle decreases water flow velocity, increasing the risk of sedimentation and eventual blockage.

Trench slopes should be designed to be practical and effective. For flow rates below 1 liter per second, the slope should not be less

than 1:40 for pipes with a diameter of 100 millimeters. For flow rates exceeding 1 liter per second, a slope of 1:80 is recommended for 100-millimeter diameter pipes and 1:150 for 180-millimeter diameter pipes.

Sharp turns should also be avoided unless inspection chambers are installed to enable monitoring and maintenance. Moreover, the entire surface should be graded to prevent water accumulation.

3.3.3.4 Waterproofing and Insulation

Waterproofing refers to measures taken to prevent water from entering the project area. During the construction phase, waterproofing can be challenging and limited. The most effective methods during this phase typically involve constructing integrated concrete (diaphragm) walls or steel (sheet) walls, which are standard in conventional top-down construction.

In the SPW approach to top-down construction, however, the peripheral concrete wall is built incrementally from top to bottom, rendering it ineffective as a waterproofing solution. Alternative methods during the construction phase are necessary here, including injecting cement or chemical solutions to fill the voids between soil particles, thereby reducing its permeability.

Waterproofing during the operational phase requires different methods. Impermeable materials such as smooth polyethylene liners (geomembranes) and textured polyethylene liners (dimple sheets) are commonly used to provide permanent insulation. However, when the SPW approach to top-down construction is utilized, greater care must be taken with these solutions due to the sequential nature of construction, which results in numerous joints that need to be effectively sealed.

According to the most widely accepted standards, including BSI, waterproofing methods can be categorized into three groups based on their installation position relative to the structure and the source of water, as detailed below.

3.3.3.4.1 Waterproofing Types Based on Position

3.3.3.4.1.1 Positive Side Waterproofing

Positive side waterproofing systems are installed on the exterior (wet side) of a structure. They shield both the interior and the structure itself from water and dampness, provided there is suitable access to the exterior of the structure. However, their maintenance and repair can be difficult, potentially reducing their durability (Figure 3.121). Most commercially available waterproofing products are designed for positive side applications, but they are unsuitable for top-down construction due to the limited exterior access it provides.

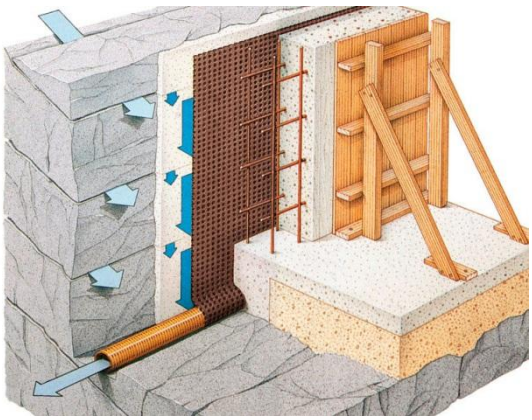


Figure 3.121 Positive side waterproofing on a floor slab and an underground wall

3.3.3.4.1.2 Negative Side Waterproofing

Negative side waterproofing involves systems installed on the interior (dry side) of a structure. In urban projects, where accessing the exterior (wet side) is financially or logistically unfeasible, these systems are necessary, though they leave some structural elements exposed to the adverse effects of water seepage, including corrosion. However, maintenance and repair are more straightforward in negative side systems (Figure 3.122). Cement-based products are commonly used for these applications. Negative side waterproofing is well-suited for top-down construction.

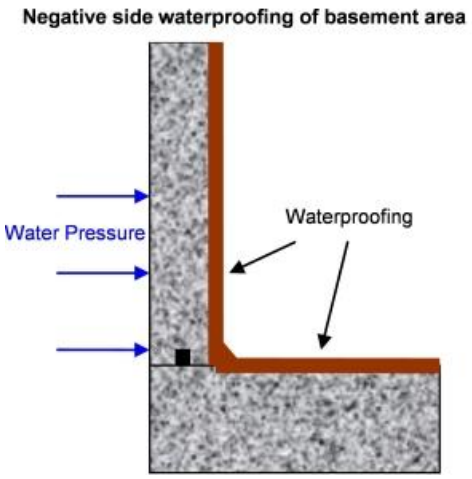


Figure 3.122 Negative side waterproofing on an underground wall

3.3.3.4.1.3 Blind-Side Waterproofing

In scenarios where a waterproofing system must be installed on the positive side but cannot be applied after concrete pouring for walls or foundations, the system should be installed before the concrete pouring process (Figure 3.123). In the SPW approach to top-down construction, this method is implemented during the incremental construction of each structural element. Here, the operational sequence typically involves: 1) waterproofing, 2) reinforcement installation, and 3) concrete pouring. A key characteristic of this method is the presence of numerous joints in the waterproofing layer, which is also its primary drawback.

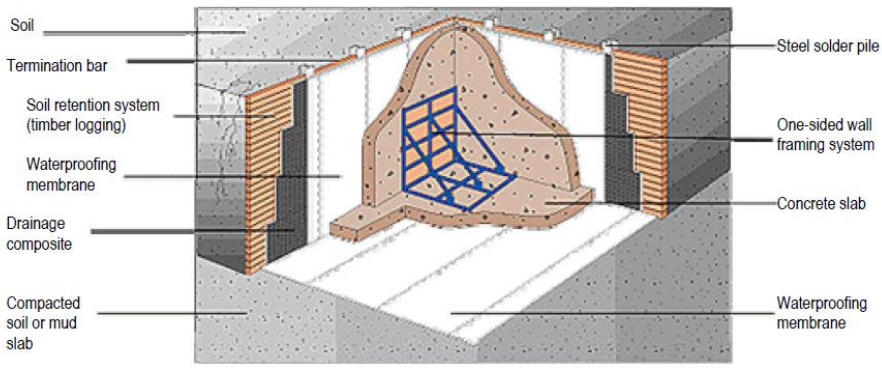


Figure 3.123 Blind side waterproofing

3.3.3.4.2 Waterproofing Types Based on Material

3.3.3.4.2.1 Insulation Cover Waterproofing

When insulation covers are used, the structure itself is not treated with additional measures to prevent water or steam permeation; instead, the waterproofing system operates independently through impermeable covers installed on interior, exterior, or within the structure. In recent years, impermeable covers have gained popularity due to their cost-effectiveness and are regarded as the best waterproofing solution available, as long as the structure's exterior is accessible. These covers can be made from bonded sheet membranes (a type of geomembrane) or geosynthetic clay liners (GCL). Other commonly used materials include liquid-applied membranes, mastic asphalts, cementitious crystallization slurries and powders, and cementitious multi-coat render toppings and coatings.

3.3.3.4.2.1.1 Bonded Sheet Membranes

Bonded sheet membranes are geosynthetic materials widely used for waterproofing. Their popularity stems from their cost-effectiveness and time efficiency compared to traditional covers including clay- and cement-based solutions. Thanks to their consistent quality and superior impermeability—far exceeding that of gravel- and soil-based solutions—they have become a staple in construction projects over the past three decades. Geomembranes feature impermeability levels 100,000 times greater than clay and 10,000 times greater than concrete, making them virtually impermeable against liquids. Additionally, sheet membranes demonstrate significantly higher durability compared to concrete, which can deteriorate under the influence of alkaline aggregates in the presence of water.

The wide variety of sheet membranes available, combined with their versatility, makes them the preferred choice over other geosynthetic materials. These membranes can be categorized in various ways. Among the most common are polymer-based and bituminous membranes, with polymer-based membranes further divided into subcategories, as shown in Table 3.2.

Bituminous geomembranes are typically prefabricated and consist of synthetic or organic fibers available in both woven and nonwoven forms, both of which offer excellent puncture resistance. These fibers are impregnated with polymer-modified bitumen to enhance their performance. Prefabricated bituminous geomembranes (PBGMs) serve a variety of purposes, including insulation for roofs, roads, channels, and pools.

Table 3.2 Geomembrane Types

Type	Name	Abbreviation
Bitumen-based	Prefabricated bituminous geomembrane	PGBM
Polymer-based	High-density polyethylene	HDPE
	Linear low-density polyethylene	LLDPE
	Polyvinyl chloride	PVC
	Flexible polypropylene	FPP
	Ethylene propylene diene monomer	EPDM
	Chlorosulfonated polyethylene	CSPE

The Iranian Institute of Standards and Industrial Research (ISIR) has established regulations for various types of prefabricated bituminous water insulation. These include:

- **Standard 3864**: Covering foundation insulation.
- **Standard 3884**: Covering the characteristics of prefabricated bituminous polymer-modified water insulation for foundations.

- **Standard 3885-1**: Covering prefabricated polymer-bituminous water insulation for roofs.

Additionally, the design and application of these products should adhere to international standards, including ASTM D2643-08.

For top-down construction projects, installing bituminous sheet membranes requires the initial application of a layer of shotcrete or smooth cement grout to the shaft wall surface. This initial layer has no load-bearing capacity and can increase project costs, but it is necessary as a base for the membrane.

The advantages and disadvantages of using bituminous sheet membranes, particularly in top-down construction, are outlined below.

Advantages:

- Easy joint execution and sealing
- Low weight (4 kilograms per cubic meter) and suitable for underground applications
- High resistance to decay and breakage
- Excellent flexibility
- Durability due to the polyester layer, providing resilience against structural expansions and contractions
- Wide availability
- Complete waterproofing when installed properly

Disadvantages:

- Requirement of a cement foundation on the excavation shaft surface, which must be free of protrusions, debris, and moisture
- Susceptibility to punctures during concrete cover installation

- Lack of a reliable quality control method to ensure waterproofing; effectiveness heavily depends on the installer's expertise

Another type of waterproofing material is the polymer-based sheet membrane. It consists of thin layers with low permeability, made from a combination of synthetic elastomers or plastomers. They can be installed either by on-site application or prefabricated through methods such as paving or heating. These membranes are primarily used to line and cover liquid and gas storage facilities, serving as a barrier against gas and liquid penetration. While not entirely impermeable, polymer-based sheet membranes offer significantly greater impermeability compared to geotextiles and most soil types, including clay soils. Figure 3.124 illustrates the use of polymer sheet membranes for foundation waterproofing.

Today, polyethylene sheet membranes are more commonly used than other types. These membranes offer exceptional chemical resistance, making them suitable for exposure to industrial wastewater and various corrosive substances. During production, chemical additives are incorporated into raw materials, enhancing their resilience and preventing damage from chemical exposure. Their chemical resistance and long-term durability are rigorously tested in compliance with American Society for Testing and Materials (ASTM) standards. Additionally, polyethylene membranes are physically flexible and exhibit excellent tensile strength, further contributing to their versatility and reliability.

Various types of sheet membranes can serve as the primary waterproofing layer, provided they possess adequate mechanical properties and durability. Notably, low-density polyethylene (LDPE) membranes, available in linear low-density (LLDPE) and very low-density (VLDPE) variants, offer an optimal balance of flexibility and durability. In top-down construction projects, these membranes are applied to soil walls at excavation shafts before concrete is poured.

The primary advantages and disadvantages of polymer-based sheet membranes, widely used in top-down construction, are as follows:



Figure 3.124 Example of geomembrane covers installed adjacent to piles before foundation execution

Advantages:

- Exceptional flexibility, enabling effective coverage of angles, corners, curvatures, and rough surfaces at the construction site
- High resistance to rupture, puncture, and abrasion
- Availability in various thicknesses, ranging from 0.75 millimeters to 2 millimeters
- Low weight, facilitating easy, cost-effective transportation and installation
- Resistance to chemical, biological, and mechanical factors in the environment
- Simple and reliable joint connection processes
- On-site repairability of potential ruptures without requiring specialized expertise

Disadvantage:

- Requiring a properly prepared base before installation
- Requiring specific construction methods to protect membranes during the construction process

3.3.3.4.2.1.2 Geosynthetic Clay Liners

Geosynthetic Clay Liners (GCLs) are a type of composite geotechnical material, often referred to as geocomposites. They consist of two polypropylene textile layers coated with high-friction materials. These layers enclose a minimum of 4,500 grams of sodium bentonite per square meter. The two textile layers are securely bonded through longitudinal stitching, creating a unified structure that effectively locks the sodium bentonite in place, preventing it from shifting or dispersing.



Figure 3.125 Geosynthetic clay liners

When exposed to water, GCLs form a uniform, cohesive layer of compacted membrane that effectively blocks water infiltration. This functionality relies on the expansive properties of bentonite, which swells upon contact with water, filling any voids and forming a durable waterproof barrier. This membrane maintains its integrity throughout the structure's lifespan. One of the key advantages of GCLs is their self-healing capability, which ensures long-term reliability.

3.3.3.4.2.1.3 Liquid Waterproofing Systems

Liquid waterproofing systems comprise a soluble blend of urethane, rubber, plastic, vinyl, and polymer asphalt. These mixtures are applied directly to surfaces in liquid form. After their curing duration, they create a waterproof barrier. Because they are applied in liquid form, ensuring a uniform thickness is essential. Typically used as positive-side waterproofing systems, they require a protective layer after backfilling is completed. In top-down construction, liquid waterproofing systems can only be utilized when a waterproof structure, such as watertight concrete, is in place, allowing for negative side application and moisture control, provided sufficient overhead pressure is provided through the internal lining.

3.3.3.4.2.2 Integral Waterproofing Systems

The core principle of these systems lies in providing water resistance by integrating waterproofing into the mix design and material selection of the structural elements themselves. The most widely adopted approach within this category is the use of watertight concrete.

Concrete mix design can be optimized to minimize water penetration. A high-strength, durable concrete mix that is properly compacted and cured can offer significant resistance to water infiltration, with this resistance largely influenced by the thickness of the concrete cross-section. However, water seepage in concrete commonly occurs at cold joints—frequent in top-down construction—as well as at utility entry points. The following considerations are crucial when designing low-permeability concrete mixtures:

All aspects of the concrete mix—including materials like aggregates, admixtures, and water—along with workmanship during pouring and curing, site management, environmental conditions, storage practices, construction sequencing,

reinforcement and connection preparation, and crack prevention—must comply with current concrete standards.

While various building codes outline the criteria for classifying concrete as waterproof, current Iranian concrete regulations lack a dedicated section on this subject, necessitating reference to other standards. The following considerations are essential for ensuring the effectiveness of waterproof concrete:

- Design, calculations, and construction details must align with the desired level of waterproofing. Proper reinforcement distribution is crucial to minimize the risk of cracking. Additionally, the placement of waterstops should be carefully considered when selecting the construction method.
- The chosen concrete curing method must align with the specific requirements of waterproof concrete.
- Execution and site conditions must align with the manufacturer's instructions to guarantee the concrete's waterproofing performance.
- Joint preparation, formwork execution, and connection establishment between formwork sections or reinforcement bars should adhere to the specified technical requirements.
- Cracks resulting from structural settlement should be carefully managed to prevent excessive widening.

Figure 3.126 illustrates an example of waterproof concrete used in a foundation.

Various admixtures, such as hydrophobic chemicals, ultra-fine solid particles, and crystallizing agents, can be used to reduce concrete permeability. Additionally, waterstops can be used at joints. A bentonite-based waterstop is a modern, easy-to-install option. Figure 3.127 illustrates both a schematic overview and a real-life example of bentonite waterstops.

In top-down construction, achieving all these conditions can be challenging, and in practice, ensuring completely waterproof concrete is often difficult.

Although using waterproof concrete is important, relying on it as the sole method of waterproofing is not recommended due to its execution and curing challenges as well as the presence of numerous joints. Waterproof concrete should be used in combination with other waterproofing techniques for optimal results.



Figure 3.126 Waterproof concrete used in a foundation



Figure 3.127 Bentonite waterstops

3.3.4 Unforeseen Factors

Despite engineers' best efforts to consider all aspects of a project, including geotechnical specifications and underground conditions, encountering unforeseen factors during excavation is almost inevitable. Constructors must be adequately prepared to address these challenges. Qanats, sewage tanks, boulders, and subsurface urban utilities are among the more common challenges engineers might encounter.

3.3.4.1 Qanats

There are 500 main qanat branches in Tehran, 150 of which are still operational. Each main branch can have numerous subsidiaries as well. Unfortunately, no detailed map of these qanats exists, making it impossible to predict their presence in a project area. In the top-down construction method, many qanats are discovered during the shaft excavation phase. Qanats pose a significant safety hazard, especially during manual shaft excavation. In areas where the presence of a qanat is suspected, implementing additional safety precautions is essential. If a qanat is discovered within the project area, the best practice is to excavate a replacement qanat to redirect its route before excavation and construction operations begin. If a qanat is discovered inside an excavation shaft, the measures outlined in Section 3.3.3 should be implemented.

3.3.4.2 Sewage Tanks

Underground voids, such as abandoned sewage tanks, pose serious risks during the excavation phase of top-down construction projects. If not identified in advance, these concealed cavities increase the risk of accidents, especially for manual excavators who may inadvertently fall into them. Given the prevalence of sewage tanks in old buildings, both within the project area and adjacent lots, it is essential to conduct thorough field observations to locate potential hazards. Despite careful surveys, the unpredictable nature of these voids necessitates strict adherence to safety protocols throughout excavation operations. Upon encountering a sewage tank or similar underground voids, the immediate priority should be its safe and hygienic drainage. Following this, the surrounding ground must be stabilized under

the guidance of a geotechnical engineer to ensure long-term safety and structural integrity.

3.3.4.3 Boulders

The presence of boulders is another unpredictable factor that can arise during the excavation phase. In some instances, boulders remain undetected during geotechnical surveys and are only discovered once excavation is underway. These boulders are often extremely resilient, having withstood long-term erosional forces, which makes their removal both challenging and time-consuming. They can weigh between a few dozen kilograms to over ten tonnes. Encountering such boulders is particularly common in valleys and areas near ancient riverbeds filled with sediment deposits.

During the shaft excavation phase, boulders can be addressed through several methods, including:

- Crushing them with mechanical hammers
- Splitting them using the plug-and-feather technique (Figure 3.128)
- Breaking them apart with expanding non-explosive chemical agents such as Katrock, ensuring all necessary safety protocols are followed
- Adjusting design by relocating the shaft or reducing its depth

It is important to note that the use of explosives is never recommended. Additionally, during the excavation phase, hammer-equipped mechanical excavators may be employed as an alternative. In some cases, removing the boulder intact, without crushing it, can be a faster option (Figure 3.129).

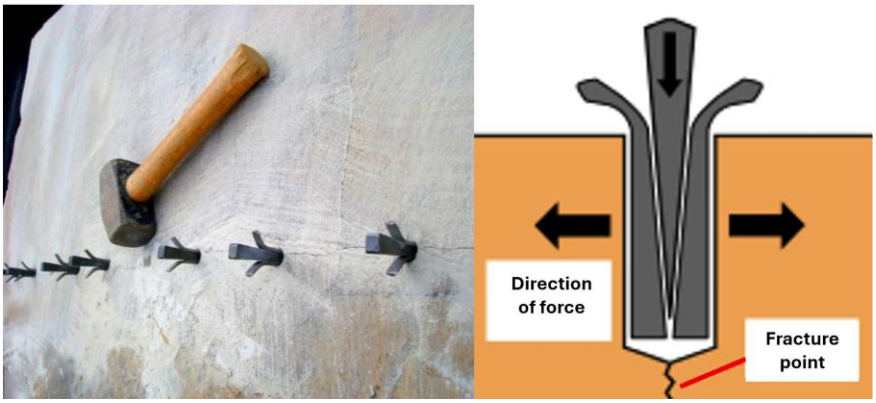


Figure 3.128 The plug-and-feather technique



Figure 3.129 Extraction of a boulder from a depth of 25 meters at a top-down construction site

3.3.4.4 Subsurface Urban Facilities

Subsurface urban utilities, such as gas, water, and sewage pipelines, as well as electrical and communication lines, represent another category of unforeseen challenges during excavation operations. In many cases, detailed maps of these utilities are either unavailable or inaccurate. To address this issue, it is essential to consult relevant organizations to identify potential locations. This step ensures legal compliance and protects the project team in case such utilities are encountered. Since top-down construction is typically confined within the project's boundaries and subsurface utilities are usually located beneath urban thoroughfares, this method causes minimal interference with urban infrastructure. The likelihood of encountering these issues in top-down construction projects is relatively low compared to other methods like anchoring or nailing.

3.4 Underground Operations Facility Requirements

3.4.1 Lighting

The primary goal of providing adequate lighting during construction is to ensure a safe and efficient environment for equipment movement and personnel activity by providing a clear, accurate, and unobstructed vision during work hours. Insufficient lighting and poor visibility can result in various safety hazards, technical issues, and discomfort for personnel during construction and excavation. Implementing a proper, standardized lighting system along with well-designed access routes is essential across all work fronts.

Lighting methods can be classified into two categories based on their positioning: general lighting and local lighting. General lighting systems are essential for facilitating common traffic within the project area, conducting routine and periodic inspections of operations, and ensuring the functionality of pipes, cables, facilities, and equipment, as well as enabling repair and maintenance. Local lighting systems, on the other hand, are designed to provide additional illumination for specific areas

where more precise visibility is necessary, including excavation sites, construction zones, and ventilation areas.

3.4.1.1 Lighting Design Standards and Regulations

Since each top-down project has unique specifications and requirements, there are no universal guidelines for lighting procurement applicable to all top-down projects. However, due to similarities between top-down construction conditions and underground construction, similar underground construction references can be consulted, including Chapter 13 of the National Building Codes, Mine Lighting Guidelines (Publication 489), Iranian National Standard #5920-1, and Publication 110-1.

The following recommendations are derived from existing technical references and are intended as general guidelines rather than mandatory requirements. It is important to note that these guidelines have not been specifically designed for top-down construction projects.

3.4.1.2 General Lighting Criteria for Underground Spaces

One of the key factors influencing lighting is the required luminance level. According to Publication 489, underground projects classify luminance into four levels based on their application (Table 3.3).

Table 3.3 Luminance levels in different working conditions

Level	Description	Luminance (cd/m^2)
1	Light traffic, low level of mechanization, general lighting for safety	0.05
2	Common use of mechanized equipment in the project area	0.2
3	Underground spaces for low-precision operations	10
4	Underground spaces for high-precision operations	20

Another critical consideration is glare from lighting equipment. To minimize glare, the luminance of lighting system components should not exceed 3000 cd/m².

3.4.1.3 Lighting Calculations

In this section, we provide guidelines for determining internal lighting levels, measured in lux (lumens per square meter). The average lighting intensity is denoted as E_{av}. When using calculation and design software, the output typically includes average lighting intensity (E_{av}), minimum lighting intensity (E_{min}), maximum lighting intensity (E_{max}), and lighting uniformity (E_{min}/E_{av}).

The lumen method is commonly employed to calculate the required lighting intensity. Its primary goal is to determine the number and placement of lighting sources to achieve the desired average lighting intensity. This method focuses solely on average lighting intensity for a given area and does not account for variations in lighting intensity across different points. If analyzing lighting intensity fluctuations from one point to another is necessary, the point-by-point method should be used instead.

The illumination reaching a work plane from a lighting source depends on several factors: the light distribution curve of the source, the dimensions of the area, and the reflection coefficients of the walls and ceilings. This is because light from a source reaches the work plane both directly and indirectly through reflections off ceilings, walls, or after multiple reflections. The E_{av} is calculated using Equation 3.4.

$$E_{av} = \frac{\varphi\, C_1\, C_2\, C_3}{XB} \qquad (3.4)$$

Where:

φ: Luminous flux of each lighting source in lux
E_{av}: Average lighting intensity on a given surface in lux
C_1: Utilization factor
C_2: Maintenance factor

C_3: Absorption factor
X: Distance between light sources in meters
B: Width of the specific surface in meters

Additionally, a portion of the light is absorbed by the lightbulb or reflector, and the remaining portion exits the light source. The light distribution curve of the exiting light can be adjusted, and this is a critical factor in lighting design.

Table 3.4 Minimum lighting level values for various workspaces

Space	Minimum lighting level (lux)
Shaft floor area	60
Critical points in the shaft's surrounding area	30
Underground maintenance station	50
Transport corridors	10
Excavation workshops	15
Loading points	20
Equipment area	25
Personnel walkways	30
Advancements	20

The utilization factor is defined as the ratio of the effective luminous flux reaching the work plane to the total luminous flux emitted by the light source. It depends on several factors, including:

- Light absorbed by the light source,
- The light distribution curve,
- The installation height of the light source,

- The dimensions of the work plane, and
- The reflection coefficients of the ceiling, walls, and floor.

A light source that directs most of its light downward typically has a higher utilization factor compared to one that disperses light toward ceilings and walls.

The influence of room dimensions and light source height can be quantified using the room index parameter:

$$K_r = \frac{LW}{h\,(L + W)} \qquad (3.5)$$

Where:

L: Length of the room (in meters)
W: Width of the room (in meters)
H: Height of the light source above the work plane (in meters)

Publication 489 provides utilization factors for various light sources based on different room indices, as shown in Table 3.5.

Depending on the cleanliness of the installation site and the type of lighting fixtures in terms of dust accumulation, three types of maintenance factors—good, moderate, and poor—are specified according to Table 3.6. In industrial settings and under normal atmospheric conditions, where the lamp bulbs are thoroughly cleaned every 6 weeks, a maintenance factor of 0.8 is typically assumed.

Depending on the height of the light source above the work plane and the level of pollution present, a correction factor should be applied in top-down projects. For dusty conditions typical in top-down construction sites, the recommended correction factor is 0.5.

Table 3.5 Utilization Factors of Light Sources for Different Room Indices

Room index	High-reflection bulb	Divergent bulb	Fluorescent light
0.6	0.33	0.16	023
0.8	0.4	0.22	0.28
1.0	0.45	0.26	0.32
1.25	0.49	0.29	0.35
1.5	0.53	0.32	0.38
2	0.57	0.36	0.41
2.5	0.6	0.4	0.43
3	0.62	0.43	0.44
4	0.64	0.46	0.46
5	0.66	0.48	0.48

3.4.1.4 Lighting Equipment Technical Specifications

Incandescent, gas-discharge, and fluorescent lamps are commonly used as light sources for general lighting. To protect these lamps against physical damage, they should be equipped with thick bulbs and a protective metal cage. Additionally, fluorescent lamps should be designed to automatically disconnect from the power supply in the event of a fault. To ensure protection against dust and other environmental hazards, manufacturers must adhere to the specifications outlined in Standard 5920.

For general lighting in short walkways, fluorescent lamps are recommended. Mercury-vapor and sodium-vapor lamps can be used if the installation height is at least 2.5 meters. Incandescent lamps, on the other hand, are suitable only for signage and as point light sources, as discharge lamps are not ideal for these

purposes. Each light source must be capable of operating continuously for a minimum of 10 hours, and luminance levels should not drop below 20 lumens after 8 hours of use.

Table 3.6 Maintenance factor for different environmental conditions

Environmental conditions	Cleaning frequency	Maintenance factor
High dust	Monthly	0.8
	Every 6 months	0.6
	Annually	0.4
	Only when replacing bulbs	0.3
Moderate dust	Monthly	0.9
	Every 6 months	0.8
	Annually	0.7
	Only when replacing bulbs	0.6

The choice of light source for each location should be based on the specific characteristics of that site. Factors such as the positioning of electrical equipment, the presence of dust, dampness, water seepage, wind speed, and wind direction must all be considered when selecting a suitable light source with adequate protection against environmental conditions. Since top-down construction sites are often exposed to dust and pollution, a minimum solid particle protection level of 5 is required. Additionally, as water seepage and discharge are unlikely, a liquid ingress protection

level of 2 should suffice. Consequently, an IP52 rating is recommended for light sources used at top-down construction sites.

3.4.1.5 General Considerations in Lighting System Design

High-efficiency light sources (measured in lumens per watt) suitable for each space and work area should be utilized. It is recommended to select lamps with long lifespans and appropriate color temperatures and color rendering indices tailored to the specific requirements of the space. To ensure adequate illumination, lamps with high efficiency and suitability for the intended environment should be prioritized.

During the lighting design phase for a space, the glare produced by each light source should be carefully considered, and lamps with minimal glare should be prioritized. Additionally, ensuring that the required light intensity is provided for a work front or work plane (work and activity areas) is essential. The design should focus on delivering task-specific illumination for each area.

3.4.2 Ventilation and Fresh Air Supply

In top-down construction, underground work areas often face significant ventilation challenges due to their limited openings and the presence of multiple ceilings above them. The operation of heavy machinery in these confined spaces generates exhaust gases, and the presence of personnel necessitates a continuous supply of fresh air. To ensure a safe and comfortable working environment, it is essential to address these challenges carefully. As outlined in Chapter 12 of the National Building Codes, measures must be taken to supply fresh air for personnel, remove exhaust gases from machinery, and control dust levels. By doing so, construction teams can mitigate health risks, enhance visibility, and create conditions suitable for efficient operations.

3.4.2.1 Ventilation and Fresh Air Supply Design Standards and Criteria

Similar to lighting design, there are no specific guidelines tailored to ventilation and fresh air supply for top-down construction. However, given the similarities between top-down and underground working conditions, Chapter 12 of the National Building Codes can serve as a reference. Additionally, the underground air supply standards developed by the Swiss Association for Standardization (SNV) may provide valuable guidance. The following recommendations are drawn from existing technical resources and should be understood as general suggestions, not requirements specifically designed for top-down construction.

3.4.2.2 Ventilation Methods

The fresh air supply in top-down construction can be implemented in two ways: introducing fresh air (positive ventilation) and extracting pollutants (negative ventilation). In a positive ventilation system, fresh air is drawn into a fan, pressurized, and directed into a ventilation duct. As the air travels through the duct, its pressure decreases due to friction with the duct walls, and a portion of the air may escape through joints and connections. The remaining low-pressure air is released at the duct's exit, where it is directed toward the work area. There, it dilutes pollutants in the work environment before exiting through floor slab openings.

Positive ventilation advantages:

- The air at the work plane is consistently fresh and healthy.
- Cloth ducts without retaining rings can be used, allowing for faster, easier installation and lower costs compared to rigid ducts.
- The fresh air entering the fan and ducts is free of moisture and dust, which enhances the longevity of the equipment.
- The fan is positioned at a fixed point outside the tunnel, making maintenance more convenient.

- Noise pollution is eliminated at the work plane.

Positive ventilation disadvantages:

- Since the system relies on diluting pollutants, a significant volume of fresh air is required, resulting in higher operating costs.

In a negative ventilation system, a fan is placed near pollution sources (e.g., machinery exhausts) or is connected to a rigid duct located close to pollution sources. The fan draws in and removes the surrounding air, creating negative pressure in the environment. This negative pressure causes fresh air to flow in through openings in the floor slab. However, this method can be challenging, as the fan must be constantly relocated as excavation advances.

Negative ventilation advantages:

- If the pollution source is stationary and localized, pollutants can be extracted before they disperse into the environment. This eliminates the need for air dilution, reducing the required volume of fresh air and lowering operational costs.

Negative ventilation disadvantages:

- Rigid ducts are required, and these must be installed within the work plane; their installation and relocation can be challenging.
- Noise pollution may occur at the work plane due to the proximity of the fan.

- Pressure drops in ducts equipped with metal rings are more significant compared to other ducts, which can increase operational costs.
- Ring-equipped ducts and rigid ducts are more expensive than flexible cloth ducts.
- Dust and moisture can pass through the fans and ducts, leading to increased equipment wear and the need for repairs.
- Non-concentrated pollutants, such as exhaust fumes from machinery, are likely to accumulate at the work plane.

3.4.2.3 Recommended Ventilation Practices for Top-down Construction

The choice of ventilation method primarily depends on the type of pollutant and its dispersion pattern. Negative ventilation is more effective when pollutants are localized and confined to a specific area, as it prevents their spread. Conversely, if pollutants are dispersed throughout the work plane, positive ventilation is the better option. By ensuring a high airflow rate, positive ventilation quickly reduces pollutant density, creating a healthier and more comfortable environment for personnel.

In tunnel construction, particularly in low-height tunnels, positive ventilation is generally more straightforward and cost-effective to implement. However, since top-down construction differs from traditional tunnel projects with its nonlinear operations, positive ventilation alone may not be effective, necessitating a combination of positive and negative ventilation systems to achieve optimal air quality and pollutant control.

Since machinery is the primary source of pollution and its non-stationary nature causes pollutants to disperse throughout the work environment, a positive ventilation system is essential in most cases to introduce fresh air. Additionally, due to the vertical progression of excavation operations and the fact that machinery-generated pollutants are often dense and tend to settle, a negative

ventilation system is also necessary to effectively extract these pollutants.

To ensure adequate air circulation, fresh air should be introduced through a vertical duct positioned at the operational level. Furthermore, for each work plane at a given phase, a horizontal duct at the operational level where workers are present is required to maintain air quality. Polluted air must also be removed through appropriately placed ducts to establish proper airflow throughout the site. Typically, in top-down projects, soil removal opening positions, due to high machinery traffic, are areas with the highest concentration of pollutants. Removing the polluted air from these points would be most effective.

3.4.2.4 Determining the Required Fresh Air Volume

The following equations can be used to calculate the fresh air volume required at a given work plane.

The air required for personnel's breathing:

$$Q_{ap} = 6 * n \left(\frac{m^3}{min}\right) \quad (3.6)$$

The air required to dilute machinery fumes:

$$Q_{adilute} = P * [36 \dots 48] \left(\frac{m^3/min}{kw}\right) \quad (3.7)$$

The air required to maintain suitable working conditions:

$$Q_{aminv} = A_s * 0.5 \left(\frac{m^3}{s}\right) \quad (3.8)$$

The air required to dilute dust:

$$Q_{adust} = \frac{(excavated\ tonnes) * 1800}{10 * t} \quad (3.9)$$

Where:

n: The number of workers active at a given work plane
P: The power of machinery active under a floor slab (in kW)
A: The work plane's cross-sectional area (in m^2)

Since the dilution of each pollutant is calculated to ensure breathable air, the highest value derived from these calculations should serve as the basis for selecting appropriate fans and ducts during the design phase.

It is important to note that fans used during the construction phase do not necessarily have to be temporary; they can also be used during the operational phase, provided the utility engineer is consulted during the fan selection process. Additionally, compliance with standards outlined in Chapter 12 of the National Building Codes is required when operating according to the guidelines mentioned above.



Figure 3.130 A positive ventilation system

Chapter 4

Key Principles of Analysis, Design, and Control in Top-down Construction

Chapter 4: Key Principles of Analysis, Design, and Control in Top-down Construction

4.1 Introduction

This chapter explores the core principles of analysis, design, and control in top-down construction projects. The top-down method requires meticulous consideration of operational, geotechnical, and structural factors—some of which are unique to this approach, while others align with standard practices. The inherent complexities of top-down construction, combined with the soil-structure interactions it entails, call for tailored design strategies and careful selection of geotechnical parameters. A robust design procedure for top-down construction must incorporate executional steps into modeling and analysis to fully account for their influence on the project. Accordingly, this chapter will outline the guiding principles of top-down design, offering a comprehensive framework to aid project planning and execution.

The discussion will provide an overview of the essential geotechnical surveys and the laboratory and in-situ tests required to initiate top-down project design, alongside practical recommendations for these activities. Following this, design considerations will be addressed in three key areas: a) execution steps and methods, b) structural analysis and design, and c) geotechnical analysis and design.

4.2 Information Required for Design

4.2.1 Prerequisites

Successful design and execution of a structure using the top-down method require comprehensive information, encompassing executional, geological, geotechnical, subsurface, hydrological, and structural details.

Top-down design involves three key stages: designing execution steps and methods, conducting geotechnical analysis and design, and conducting structural analysis and design. The process begins with designing execution steps and methods, starting with the preparation of a conceptual execution plan and the definition of primary execution criteria and steps. This is followed by geotechnical modeling, analysis, and design. Finally, structural analysis and design are carried out. It is important to note that revisiting earlier stages may be necessary to make adjustments and ensure a cohesive design.

The prerequisites for designing execution steps and methods include architectural details, the site plan, an assessment of environmental and equipment limitations, employer requirements, the presence or absence of underground water, and any other operational challenges. While some of these prerequisites remain consistent across different projects, others can vary significantly depending on the specific characteristics of a project and the design process.

Structural design prerequisites can be categorized into two main parts: general structural design requirements and those specific to the selected execution method. The latter includes factors such as construction steps, structural conditions during the construction process, and considerations derived from geotechnical analysis and design, such as soil pressure on the structure.

Given its inherent complexity, geotechnical analysis and design demand a broader set of prerequisites. A comprehensive geotechnical investigation for a top-down construction project requires appropriate tools and methodologies to accurately determine project specifications. These specifications serve as the foundation for planning, designing, and constructing the structure, identifying potential construction risks, and providing realistic cost and time estimates.
At a minimum, a geotechnical investigation should evaluate the stability of wells and shafts and excavation fronts, the structural

design of the top-down method, the effects of excavation and drilling on surrounding structures and facilities, underground water conditions, and the short- and long-term impacts of shaft construction on these factors. Additionally, it should assess the potential execution risks. The scope of geotechnical investigations must align with key project characteristics, such as location, priority, hazard levels, service life, scale, budget, constraints, constructability, and the influence of external parties.
Geotechnical investigations preceding top-down excavation operations typically include the following steps:

- Collecting and reviewing existing data
- Conducting in-situ and laboratory tests and investigations
- Performing geophysical tests
- Carrying out environmental studies
- Evaluating underground water conditions

Before initiating subsurface investigations or conducting in-situ and laboratory tests, basic information must be collected. This includes published and unpublished data about the project site and surrounding areas, existing images of the location, and detailed investigations of the area.

As part of the investigation, it is crucial to gather data on the design and construction of adjacent structures. Additionally, local knowledge from experienced residents or professionals about past projects in the vicinity may offer valuable insights that may be inaccessible as published texts.

Geotechnical investigation planning should be entrusted to an experienced geotechnical consultant well-versed in the design and construction of top-down projects to ensure the process is thorough and effective.

4.2.2 Subsurface Exploration and Geotechnical Investigation

Subsurface exploration in a geotechnical investigation program involves borehole drilling, conducting in-situ tests—such as

Standard Penetration Testing (SPT)—collecting both disturbed and undisturbed soil samples, and assessing underground water conditions.

4.2.2.1 The Number of Boreholes and Their Position

Borehole drilling is essential for identifying subsurface layers and collecting disturbed or undisturbed samples for visual classification and laboratory testing. To develop an accurate subsurface profile at a given location—assessing factors such as depth variations and soil stratification—and to evaluate geotechnical risks affecting shaft design, construction, and performance, a sufficient number of boreholes must be drilled at appropriate depths. To ensure reliability, the geotechnical investigation program should adhere to the guidelines outlined in Chapter 7 of the Iranian National Building regulations or equivalent standards.

Given the unique requirements of top-down construction, one of its most critical components is shaft excavation and pile construction, typically performed manually. To evaluate conditions for manual shaft excavation and conduct in-situ tests at specified depths, it is recommended to establish at least three test pits at the project site at the foundation depth. These test pits help assess excavation efficiency, identify technical challenges, and facilitate in-situ testing. If underground water is present, appropriate measures for manual excavation, redirection, and drainage must be implemented in accordance with the principles outlined in Chapter 3 of the present guideline.

The final number of boreholes is determined based on subsurface conditions and any challenges encountered during the drilling process. Additional boreholes may be required to identify and assess localized risks, such as qanats, shafts, unstable soil layers, or factors contributing to soil destabilization, as well as to evaluate their potential impact and implement protective measures for structures and facilities. Moreover, for sites near sensitive structures—such as historical landmarks or bridges—it is essential

to ensure the availability of thoroughly documented borehole data.

4.2.2.2 In-situ Tests

In-situ tests are conducted to determine the strength and deformation properties of soil, providing essential data for design and analysis. These tests should adhere to relevant standards, such as ASTM or AASHTO, as well as Chapter 7 of the Iranian National Building regulations. It is recommended to perform in-situ tests near geotechnical boreholes to ensure consistency in soil classification at each location and to maintain coherence between in-situ and laboratory test results. In-situ testing can be carried out using mechanized boreholes, test pits, manually excavated exploration galleries, or geophysical methods. Each of these approaches will be discussed in the following sections.

4.2.2.2.1 In-situ Tests Carried Out in Mechanized Boreholes

One of the most commonly used in-situ tests is the Standard Penetration Test (SPT), which serves as a general-purpose test in Iran and worldwide. This test is primarily conducted to evaluate the stiffness of subsurface layers and, if necessary, to collect soil samples. Performed within a mechanized borehole, the SPT must comply with the guidelines outlined in Chapter 7 of the Iranian National Building regulations and Publication 224 issued by the Planning and Budget Organization of Iran or equivalent standards. It should be conducted in all boreholes at intervals of no more than 2 meters during excavation operations.

In addition to SPT, other mechanized in-situ tests are recommended depending on the soil type and the importance of the structure being constructed. These include the Cone Penetration Test (CPT), which is a self-penetrating test; and the Pressuremeter Test (PMT), which is conducted within pre-drilled boreholes. Various types of in-situ tests, along with their key characteristics, are summarized in Table 4.1. The selection of the type and number of these tests should be guided by the designer's requirements and aligned with the geotechnical engineer's recommendations, ensuring suitability for specific soil conditions.

4.2.2.2.2 In-Situ Tests Carried Out in Test Pits

In small urban projects where mechanized in-situ tests may not be financially viable, manually excavated test pits can be used for in-situ testing. This approach is particularly common in urban areas with low groundwater levels. The most significant tests conducted in such scenarios are the Plate Load Test (PLT) and the Direct Shear Test (DST). These tests should be performed under the supervision of a geotechnical engineer and aligned with design requirements at various depths.

The PLT, as well as its interpretation, should adhere to the guidelines outlined in Publication 736 by the Planning and Budget Organization of Iran or equivalent standards. It can also be conducted within horizontal galleries, with specific locations determined by the geotechnical engineer. Similarly, the DST can be performed at varying depths in manually excavated pits, following relevant standards and based on recommendations provided by the consultant engineer.

In scenarios where the groundwater level is low or can be lowered effectively, conducting at least three DSTs and three PLTs at varying depths is recommended. These tests, carried out under the guidance of a geotechnical engineer, provide a comprehensive understanding of the soil's mechanical properties, contributing to the design and financial optimization of top-down projects.

Table 4.1 In-Situ Soil Tests According to ASTM and ASSHTO Standards

Test	Procedure	Soil Type	Soil Properties	Limitations/Considerations
Cone Penetration Test (CPT)	A cylindrical probe, driven vertically into the soil under hydraulic pressure, measures the resistance at the cone tip and along the steel body of the probe. Measurements are typically taken at intervals of 1 to 2 inches.	Silt, sand, clay, peat	Estimating soil type and detailed stratigraphy: • Sand: $\sigma'_{h\,o}$, D_r , φ' • Clay: σ'_p , su	A suitable tool for obtaining a continuous profile of soil characteristics, even allowing for the detection of thin layers that cannot be identified by conventional SPT tests. In this test, no soil sampling is performed, and if the test is conducted in sandy soils, it may lead to damage to the probe.
Piezocone Penetration Test (CPTu)	Similar to CPT, with the addition that pore water pressure is measured using an electric pressure gauge and a porous filter."	Silt, sand, clay, peat	Similar to CPT, with the following additions: • Sand: U_0 • Groundwater level • Clay: *OCR, ch, kh,* σ'_p	If the filter and inlets are not fully saturated, the measured pore pressure values may be misleading. Compression and surface wear of the interface (u_l) may affect the readings, particularly making the test results unsuitable for estimating the soil's deformation properties.

Seismic CPTu (SCPTu)	Similar to CPTu, with the addition that shear waves generated at the surface are recorded by a geophone at 3-foot intervals along the profile to calculate the shear wave velocity.	Silt, sand, clay, peat	Similar to CPT, with the following additions: • $G_{max}, E_{max}, \rho_{tot}, \varepsilon_0, V_s$	A suitable tool for obtaining the shear wave velocity of soil layers, which indicates the soil's maximum stiffness and is a key parameter for estimating soil deformation characteristics at small strains and evaluating the seismic behavior of soil. To calculate the shear wave velocity, first arrival times must be used (if crossover times are used, errors in calculating shear wave velocity increase with depth).
Dilatometer Test (DMT)	A flat plate is hydraulically pushed into the soil to the desired depth. At approximately 12-inch intervals, the pressure required to expand its thin membrane is recorded. Typically, two to three measurements are taken at each depth.	Silt, sand, clay, peat	Estimation of soil type, stratigraphy, and total unit weight. • Sand: φ', E, D_r, mv • Clay: $\sigma_p', k_0, su, mv, E, ch, kh$	If the membrane inflates excessively, it may deform. Deformed membranes do not provide accurate readings. Leakage in the tubing with connections will result in high readings. This test is effective for estimating soil deformation characteristics at small strains.

Pressuremeter Test (PMT)	A borehole is drilled, and its bottom is carefully prepared for the placement of the probe. The pressure required to expand a cylindrical membrane to a specific volume at a defined radial strain is recorded.	Silt, sand (low responsivity in some cases), clay, peat	su, mv, G, E	Drilling boreholes is the most critical step for obtaining reliable results. This test provides valuable information for calculating lateral deformation properties.
Full PMT	A cylindrical probe with a pressure gauge attached to the back of a conical tip is hydraulically driven into the soil and stopped at desired intervals for testing. The required pressure for the expansion of the cylindrical membrane at a specific volume and with a defined radial strain is recorded.	Silt, clay, peat	su, mv, G, E	The disturbance caused by the penetration of the probe leads to the stiffening of the initial modulus and the uplift pressure (p_u). It provides useful information for calculating the lateral deformation characteristics.

Vane Shear Test (VST)	A four-bladed vane, which is hydraulically driven into the bottom of the borehole, is rotated slowly while the torque required to rotate the vane is recorded to calculate the maximum undrained shear strength. The vane is rotated at a speed of 10 revolutions per minute, and the torque required for soil failure is recorded to calculate the residual undrained shear strength.	Clay, some silts, undrained peat, unsuitable for granular soils	σ_p, su, S_t	The absence of disturbance in soft sensitive clays will lead to a reduction in the measured shear strength. Partial drainage in silts and fissured clays may result in errors in the calculation of strength. The friction of the drilling rod must be considered in the strength calculation. To ensure accurate measurements in various clay deposits, appropriate selection of the vane diameter and torsional moment capacity is essential.

4.2.2.2.3 Geophysical Tests

Geophysical tests should be used alongside direct geotechnical investigation methods to accurately identify and characterize subsurface materials. They help determine the vertical profile of the upper bedrock and its quality, as well as underground water depth, soil layer boundaries, organic sedimentation limits, the presence of voids, and the location and depth of subsurface utilities, foundations, and other potential obstacles or conflicts.

Moreover, geophysical tests are valuable for obtaining vertical profiles of shear wave velocity in soil and rock structures, which are essential for seismic analysis. Additionally, shear wave velocity serves as a key parameter for estimating deformation modulus at small strain levels.

4.2.2.2.4 Recommendations for Selecting Appropriate In-Situ Tests

Considering top-down design requirements and the in-situ tests discussed, the following recommendations are proposed:

- The SPT test should be conducted at various depths in all projects.
- For soft clay soils, particularly in coastal areas, CPT and CPTu tests are recommended.
- To determine appropriate soil shear resistance parameters, the DST test should be performed at multiple depths, especially where soil layers change.
- To assess soil stiffness, the PLT test should be conducted at different depths, particularly when layer changes are present; the test should be repeated as many times as needed.
- To evaluate lateral soil stiffness for defining equivalent soil springs in soil-structure interaction analysis during top-down construction modeling, the PMT test should be performed at various depths.

4.2.2.3 Laboratory Tests

Laboratory tests are essential for obtaining soil classification data and determining key design and modeling parameters. These tests should be conducted in accordance with the guidelines outlined in Chapter 7 of the Iranian National Building regulations and the standards established by ASTM and AASHTO. The most commonly performed laboratory tests for both manual and mechanized excavations, depending on soil conditions, include the following, with their type and frequency determined by the geotechnical engineer:

- Reconnaissance tests, including gradation analysis, Atterberg limits, hydrometry, compaction tests, and Proctor tests.
- Strength tests, including triaxial tests, direct shear tests, and Unconfined Compressive Strength (UCS) tests.
- Deformation tests, including consolidation tests and triaxial tests.
- Chemical tests, including chloride, sulfate, carbonate analysis, pH, electrical conductivity (EC), and other assessments of soil and pore water chemical properties.

When selecting and conducting laboratory tests, the following should be considered:

- The type of triaxial test should be determined based on soil properties and the designer engineer's judgment.
- For all tests on remolded samples, compaction, moisture content, and applied stress levels should reflect real conditions and sample depth.
- When performing direct shear tests on soil layers containing gravel, a large-scale test (30x30) must be conducted.
- Consolidation tests should only be performed on undisturbed samples.

- Shear resistance variables should be selected based on the expected level of deformation, using either peak or residual strength variables at the designer's discretion.
- Repeatability should be ensured for all tests.

4.2.2.4 Groundwater

The depth of groundwater should be recorded during the drilling of all boreholes. To monitor seasonal fluctuations in groundwater levels within the excavation area, periodic evaluations should be conducted using observation wells or piezometers. Additionally, soil permeability should be assessed via in-situ or laboratory tests.

Local water sources—such as qanats, sewage canals, or wells—should be identified. This can be achieved by consulting existing qanat maps from crisis management organizations, the Water and Wastewater Company, and other relevant administrative bodies. Local inquiries and a review of previous projects in the area can also provide valuable insights. Depending on the project's scope and significance, geophysical tests may be used to locate wells, qanat shafts, and sewage canals. Particular attention should be given to the location of the urban sewage system in relation to the project site.

The chemical properties of groundwater sources should also be analyzed in top-down projects, as corrosive water can compromise the longevity of excavation walls. If specialized concrete mixtures resistant to corrosive substances are not used, this analysis is even more critical. Factors contributing to water-induced damage include, but are not limited to, unsuitable pH levels and high concentrations of sulfate and chloride. While geosynthetic waterproofing products can reduce direct exposure of concrete walls to soil, additional protective measures are essential to ensure the long-term durability of structures throughout the project's designed lifespan.

Considerations regarding underground water include the potential effects of water table subsidence on the settlement of

nearby structures, facilities, and surrounding infrastructure; the migration of soil and groundwater contaminants due to dewatering; possible impacts on aquifers; and water seepage into the excavation site.

4.2.3 Design Parameters

Selecting appropriate design parameters for soil materials is a critical consideration. These parameters can be determined using one or a combination of the following methods:

- In-situ testing conducted during subsurface investigations, including geotechnical and geophysical tests
- Laboratory testing
- Back analysis of design parameters using data collected during the execution phase

The selection of parameters, in addition to the results obtained from the above methods, is based on broader experience, published relevant data, local expertise, and specific correlations with the geological formations of the site. When using published correlations alongside one of the aforementioned methods, the applicability of the correlation to a specific geological formation should be verified through local experience, results from local tests, or long-term observations.

The following sections will outline the mechanism for selecting various design variables.

4.2.3.1 Shear Strength Parameters

When selecting soil shear strength parameters, the following considerations should be taken into account:

- Execution loading rate relative to the soil's hydraulic conductivity (drained versus undrained resistance)
- Loading direction
- Strain level
- In-situ stress conditions in laboratory test analysis
- Impact of execution sequence

Typically, to estimate the undrained shear strength (S_u), consolidated undrained (CU) test results and unconsolidated undrained (UU) test results, and, if necessary, in-situ test results are consulted. If soil conditions make it prone to disturbance during sample collection, transfer, or preparation, in-situ testing is recommended. For the relatively thick cohesive soil deposits, the S_u profile should be presented as a function of depth to account for stress history and soil properties.

The drained shear strength parameters, φ^' and c^', in cohesive soils should be determined using slow direct shear tests, triaxial CD tests, or CD tests with pore pressure measurements. In laboratory tests, the loading rate should be slow enough to prevent full dissipation of excess pore pressure in drained or undrained conditions and to ensure complete equilibration of pore pressure throughout the sample.

4.2.3.2 Deformation Parameters

The drained and undrained elastic deformation parameters, such as Young's modulus and Poisson's ratio, should be determined using in-situ tests—including PLT and PMT—direct laboratory tests, or empirical correlations with in-situ measurements, such as SPT. In-situ tests are recommended as the primary basis for selecting deformation parameters.

4.2.3.3 Soil Spring Stiffness

During the structural design phase of the top-down construction method, soil must be incorporated into the modeling process, with careful consideration of soil-structure interactions. A common approach is to use Winkler springs to simulate soil stiffness around structural elements. The key step in this process is determining the stiffness of the equivalent soil springs, which can function laterally or vertically under different conditions alongside structural elements.

The stiffness of equivalent soil springs can be determined using various methods. This value may be defined as linear or nonlinear,

depending on soil conditions, the designer's judgment, and strain levels.

- Method 1: Utilizing PMT test results, which provide the most accurate approach for determining the lateral stiffness of soil in structural modeling around piles.
- Method 2: Applying analytical approaches to determine the stiffness of lateral and vertical soil equivalent springs, known as p-y, q-z, and t-z curves. The API (American Petroleum Institute) standard method is one of the most commonly used approaches in this regard.
- Method 3: Deriving p-y, q-z, and t-z curves through geotechnical numerical modeling. In this approach, piles are modeled within the soil, and staged lateral and vertical loading is applied to generate these curves based on the imposed forces and resulting deformations at various depths and points. During this process, designers should carefully consider proper numerical modeling principles and accurately define interactions and interfaces.

4.3 Design Framework

Typically, top-down design consists of three distinct yet interdependent phases, each sharing certain key considerations. These phases are:

- Designing execution stages and methods
- Structural analysis, design, and control
- Geotechnical analysis, design, and control

Analysis, modeling, and construction are integral to designing a top-down structure. The first phase of design involves determining the construction method, which encompasses construction stages, key considerations, and requirements, as well as details and impacts of each stage.

The design should address several critical aspects, including:

- The construction sequence of structural columns
- The method of constructing floor slabs (sequential, alternating, etc.)
- The construction of retaining walls and shear walls (if applicable)
- The execution of other structural components
- The excavation process, including its sequencing and path
- The construction timeline and sequencing of aboveground structural elements
- The construction timeline and sequencing of underground structural elements
- The excavation pace and scheduling
- The number and placement of openings for soil removal and material entry
- The selection of necessary equipment and tools

Before defining these stages, the feasibility of the project should be thoroughly evaluated to ensure the selected execution method is practical and efficient.

A structure that has been conventionally designed for the final service load combinations, incorporating all design load combinations as per relevant codes, undergoes an initial assessment of its general structural sections and the selection of its structural system. After completing the design stages for the execution method, the structure then proceeds to the modeling, analysis, and design phase for top-down construction. This process consists of two stages.

After the initial estimation of the structure's cross-section and the selection of its structural system, the construction sequence should be incorporated into the analysis and design process, ensuring that structural elements are monitored throughout all construction phases. This can be achieved by modeling staged construction in structural analysis and design software such as ETABS and SAP2000, using predefined staged construction loads. These steps should align with the construction method design principles. If

any modifications are introduced during execution, they must be communicated to structural and geotechnical engineers to evaluate their impact.

The modeling and analysis of staged construction require defining appropriate boundary conditions for embedded columns and retaining walls. Since, at all construction stages before reaching the foundation level, the structure primarily consists of columns and walls supported by individual piles—through which all construction loads, both live and dead, are transferred to the ground—it is essential to include these piles in the staged construction modeling process. To achieve this, boundary conditions should be carefully defined using linear or nonlinear springs to simulate soil behavior, based on the geotechnical designer's judgment and the depth of the equivalent soil springs.

At each construction stage, as the embedded length of columns and walls decreases and conditions change, boundary conditions must be adjusted accordingly. For each stage, these conditions should be revised based on the updated structural and geotechnical parameters to ensure that piles, along with other structural elements, are properly analyzed and designed for staged construction loads. Using this approach, once the surface foundation is established and excavation is complete, the structural analysis and design of the foundation on piles can be performed as a fully integrated model. Additionally, if there is direct contact between a retaining wall and the adjacent soil without insulation, the retaining forces generated by friction between the concrete wall and the soil can be incorporated into the design, leading to a more cost-effective solution.

In addition to structural analysis and design for staged construction—by defining nonlinear staged construction loads—geotechnical modeling and analysis must also be conducted. The geotechnical engineer should supply the structural designer with the necessary geotechnical data for structural design. These data may include soil equivalent springs (either linear or nonlinear,

derived from empirical or numerical methods), distribution patterns, and soil pressure values at various depths and adjacent areas, determined using methods appropriate for the selected construction approach.

The geotechnical engineer should also evaluate the effects of excavation and top-down construction on nearby structures, assessing soil deformations caused by the construction process. These deformations must remain within the safe, allowable limits for both aboveground and underground structures, including subsurface facilities and utilities.

Therefore, during the modeling, analysis, and design phases, construction engineers, geotechnical engineers, and structural engineers should collaborate closely to oversee top-down excavation and construction, ensuring a comprehensive evaluation from execution, structural, and geotechnical perspectives.

4.3.1 Design Principles

A structure designed for top-down construction can be steel, concrete, or composite. It is essential to select load and resistance factors based on the structural type and relevant codes and standards. These factors should be determined in accordance with Topics 6, 9, and 10 of the Iranian National Building regulations, as well as the AISC 360 regulations for steel structures and ACI 318 regulations for concrete structures.

In most top-down projects, the construction phase is relatively short (typically under two years), and structural analysis and design are primarily based on dead loads, live loads, and soil loads. However, designers should also account for potential live load increases during construction, such as material storage weights and machinery traffic, and incorporate them into the structural analysis. These construction-phase loads can be calculated using ASCE/SEI 37-14, Design Loads on Structures during Construction. If the construction phase extends beyond

two years, a seismic analysis must also be considered in accordance with Iranian National Code 2800.

For surface foundations and piles, resistance reduction factors should be selected in accordance with Topic 7 of the Iranian National Building regulations. The entire structure-foundation system can then be analyzed and designed using the Load and Resistance Factor Design (LRFD) method.

In geotechnical analysis, the allowable stress method—using safety factors outlined in Topic 7 of the Iranian National Building regulations—can be applied to ensure excavation stability and determine surface foundation and pile load capacities. Additionally, in numerical analysis, the safety factor obtained in geotechnical design represents the lowest value across the entire model; therefore, it must exceed the minimum allowable safety factor. The overall safety factor, ground settlement, and load capacities of piles and foundations should be determined in accordance with Topic 7 of the Iranian National Building regulations.

The geotechnical designer should supply the structural designer with soil-equivalent spring data, which must be integrated into the structural analysis and design. For pile load-bearing capacity and settlement management, these data (force-deformation relationships or equivalent spring stiffness) should be used in the design without applying safety factors, and deformation analysis should be conducted based on unfactored loads. When evaluating pile load-bearing capacity under tensile, compressive, and lateral loads, the induced forces in the piles can be compared with allowable values that account for safety factors.

One of the challenges in geotechnical design is the absence of comprehensive regulations for applying the LRFD method to all geotechnical aspects. Although AASHTO (2014) provides guidelines for geotechnical design using LRFD, these guidelines are not fully compatible with current structural design codes and

regulations. This incompatibility arises because the load and resistance factors in AASHTO differ from those specified in Topics 6, 9, and 10 of the Iranian National Building regulations, which govern steel and concrete structures. Given that certain aspects of geotechnical control are typically integrated into structural design, it is advisable to use the allowable stress method for evaluating foundation load-bearing capacity and related considerations.

4.3.2 Construction Method Design

The technical aspects of construction are thoroughly discussed in Chapter 3 of this guidebook. The construction method design is based on these technical considerations.

Once the feasibility of the top-down method for a project is confirmed, a detailed, step-by-step execution plan must be developed, outlining all construction phases and their sequence. Additionally, the specific execution techniques to be used should be selected and clearly defined. Key considerations in construction method design include:

- Dimensions, cross-sections, and excavation methods for wells, shafts and galleries
- Construction, transportation, installation, and placement of columns
- Concrete pouring techniques for piles
- Selection of excavation equipment and tools
- Soil removal methods and the location of openings for soil and material transfer
- Construction methods for walls and floor slabs
- On-site executional considerations and work front establishment

These factors serve as essential inputs for designers when defining execution steps in their modeling, analysis, and design process. They should be documented as execution plans and, in some cases, included as technical guidelines in the construction documentation for top-down projects.

4.3.3 Structural Analysis, Design, and Control

4.3.3.1 Structural Limit States

The following limit states should be considered for structures built using the top-down method. Depending on the chosen construction approach and the structure's geometry, some or all of these states may occur and must be accounted for. Floor slab limit states, for instance, should be evaluated in cases where large openings are present in the slabs.

- Auxiliary mid-span columns limit states
  - Buckling under weight loads
  - Flexural-axial failure under lateral loads from soil
  - Shear failure under concentrated loads at support locations
  - Flexural failure due to lateral deformations and secondary impact of axial forces

- Pile limit states
  - Differential settlement of supports
  - Column punching into the pile
  - Shear-flexure failure

- Floor slabs and column lateral support limit states
  - Out-of-plane (slab) buckling
  - In-plane shear failure around openings
  - In-plane flexural failure around openings

In addition to these limit states, other potential limit states should also be considered in the final design. Those listed here are specific to the top-down construction method. They can be addressed either through manual analytical checks based on regulations or by utilizing software solutions.

4.3.3.2 Structural Modeling, Analysis, and Design in Top-down Construction

To analyze a structure and determine the internal forces acting on its components, various modeling methods can be employed, each offering different levels of precision.

4.3.3.2.1 Manual Structural Analysis

Manual structural analysis can be a useful tool in certain cases, providing a simple and quick method to verify results obtained from software analyses. In top-down construction, manual analysis can be conducted by categorizing columns into two main types: side columns (adjacent to soil) and interior columns (positioned within the plan area).

For analysis, each column type is modeled as a continuous beam where the number of spans is the same as the number of underground floors. Floor slabs and deep foundations (piles) serve as joint supports. At each floor slab level, an axial force corresponding to the column's span load is applied. Additionally, for side columns, a lateral load is introduced based on the column spacing in the plan (representing the soil load span) and the soil pressure distributed to the structure.

In the initial phase of manual analysis, soil pressure distribution at each floor level can be approximated as a uniform average. Bending moments can be estimated using standard calculations for continuous beams subjected to discrete, constant loads.

The analysis and design of this continuous beam are conducted in accordance with the excavation and construction stages. At each stage, key factors such as the extent of excavation, the length of the column left unsupported (i.e., without anchors) under lateral soil pressure, and the resulting lateral bending loads on side columns must be carefully evaluated. Additionally, interior columns must be checked for buckling due to axial loads from overhead weight during construction. These conditions often represent the most

critical design challenges. Therefore, a thorough structural evaluation at every construction stage is essential.

The design and verification of columns' load-bearing capacities are conducted using conventional structural design methods, employing standard linear analysis and design techniques. To ensure structural integrity, axial, bending, and shear forces in columns must be carefully considered. Additionally, second-order effects of axial loads, which can amplify bending moments induced by lateral forces, should be accounted for. The interaction between bending and axial loads is particularly critical in the design of side columns.

4.3.3.2.2 2D Modeling

In 2D structural modeling, the continuous beam subjected to axial and lateral loads, as described in the previous section, is analyzed and designed using software solutions. This approach allows for a more precise calculation of soil pressure distribution. Additionally, when analyzing a representative cross-section of the structure, including interior columns and the floor slab, the slab's bending stiffness can be incorporated into the structural analysis.

Software solutions allow for the simulation of a structure's construction stages, enabling nonlinear static analysis to account for the progressive addition of elements and loads throughout the design process.

This approach enables more accurate resistance and deformation control compared to manual analysis. However, every computer-generated analysis should be validated against simpler analytical methods to ensure reliability.

4.3.3.2.3 3D Modeling

3D modeling is essential for structural analysis in the following cases:

- Complex geometry or irregular floor plans

- Large openings in floor slabs, requiring detailed force analysis within the slab
- Varying excavation depths across different work fronts, which may cause asymmetric soil pressure and lateral structural deformations

During the 3D modeling process, it is crucial to account for both in-plane and out-of-plane slab stiffness. Additionally, the common assumption of a rigid diaphragm in building modeling is not applicable in this case.

Lateral soil pressure loads can be applied directly to peripheral walls or, alternatively, distributed to adjacent columns based on their load-bearing capacity. In the latter case, retaining walls should be analyzed and designed separately.

In some cases, a structure's geometry may prevent the horizontal equilibrium of lateral soil loads. In such instances, more precise analyses are required to determine active and at-rest soil pressures on the structure, which can then be incorporated into the model. Alternatively, equivalent springs can be used to simulate at rest soil pressure in walls moving toward the soil. Regardless of the chosen approach, accurate structural modeling and thorough deformation control—based on soil conditions—are essential.

4.3.3.3 Soil-structure Interaction Modeling

To conduct modeling of soil-structure interactions, the following assumptions can be adopted. Model precision increases as more of these assumptions are adopted:

- Force-based modeling which accounts for lateral soil pressure but does not consider soil stiffness
- Equivalent spring modeling
- Continuum modeling

The first approach is the most commonly used in structural calculations. It does not explicitly model soil-structure interactions

but instead applies lateral soil pressure to the structure. Here, the structure's stiffness and deformations are not considered in the pressure distribution, and soil pressure values are typically derived from empirical equations.

In some cases, determining the exact type of soil pressure acting on a structure—whether active, at rest, or passive—along with its depth-dependent distribution, can be challenging. A more refined approach involves applying active or at-rest lateral pressure to the structure while also incorporating soil stiffness through equivalent springs. These springs should be defined to function only in compression, necessitating a nonlinear structural analysis. If the structure moves toward a soil wall, the lateral springs are activated, simulating passive soil pressure.

Although modeling the soil as a continuum provides the most reliable results for internal force calculations, it requires nonlinear analysis, is highly sensitive to the soil's mechanical properties, and necessitates specialized software. Therefore, it is not always recommended. If this method must be used, it is advisable to supplement it with simpler methods to validate the results.

A practical alternative that balances accuracy and feasibility is to first determine the soil pressure distribution using continuum modeling while taking the structure's elastic rigidity into account, then apply this refined pressure distribution in structural modeling. This approach enhances precision in soil pressure calculations while allowing for effective structural modeling in software solutions. When using this method, the effects of friction between perpendicular walls and the structure's movement should be considered to prevent excessive moments in temporary columns.

4.3.3.4 Column Buckling Control During Construction

In the top-down construction method, columns are initially installed on piles within shafts, extending continuously to the full depth of the shaft. Before the lower levels of the structure are built,

the buckling length of the column inside the shaft equals the shaft depth, significantly reducing its axial load-bearing capacity. To optimize structural performance and prevent excessive column dimensions, the buckling length within the shaft must be minimized during the construction phase, before the completion of the underground structure. As discussed in Chapter 3, this can be achieved by installing temporary supports inside the shaft or by filling the shaft with coarse aggregates or soil-cement mixtures temporarily.

If temporary supports are used, they must be securely positioned between the shaft wall and the column to restrict lateral movement. Additionally, these supports should be installed in a way that prevents displacement due to accidental impacts during construction. The spacing between supports is determined based on column buckling calculations.

An alternative approach involves filling the shaft with coarse aggregates, soil-cement mixtures, or removable plastic concrete. In this method, the column's buckling behavior depends on the stiffness of the fill material. The critical buckling load can be calculated using established equations for columns in an elastic medium. Based on the column's effective free length, as determined by the construction sequence, its load-bearing capacity is compared against permissible resistance values. Additionally, if the soil is modeled using equivalent springs, these springs should be removed from the model during excavation, ensuring that the software correctly accounts for the column's free length in the buckling analysis.

4.3.3.5 Floor Slab Structural Control

Floor slab control in top-down construction is crucial for two primary reasons: 1 out-of-plane buckling due to lateral soil pressure; and 2) in-plane shear and flexural effects around openings. While these considerations are also relevant in the final structure, they become particularly critical during the construction

phase, as floor slabs may undergo more severe conditions than in a fully completed structure.

Several factors contribute to this increased vulnerability during construction:

1. In some top-down construction scenarios, excavation steps may exceed typical floor heights, and sections of the peripheral walls may remain incomplete during excavation. In such cases, the internal compressive forces within the slab—acting as lateral supports and peripheral column anchors—can be significantly higher than those experienced in the final structure. Additionally, soil pressure distribution differs between a construction site and a completed structure.
2. To facilitate soil removal from underground floors, it is sometimes necessary to create additional openings in floor slabs during construction. These temporary openings might reduce slabs' structural integrity and can lead to excessive in-plane deformation.

Given these challenges, floor slabs must be carefully analyzed and controlled at each stage of top-down construction to ensure structural stability and performance.

To calculate in-plane forces in a floor slab, 3D structural modeling is required. A key consideration is that the rigid diaphragm assumption, commonly used in building structure modeling, is not applicable here, as it prevents the extraction of in-plane forces in the floor slab.

For modeling one-way concrete joists (ribbed slabs), a conservative approach is to assume only the in-plane rigidity of the slab on the joists. For greater accuracy, orthotropic plane elements—available in some structural analysis software solutions—can be used.

For two-way slabs (waffle slabs), membrane-bending shell elements can be employed. These elements can either be modeled with an equivalent thickness or with a detailed cross-section definition, provided the software used supports this level of detail.

Additionally, applying appropriate cracking coefficients to different structural components—based on concrete design code recommendations—can improve the accuracy of the analysis results.

If in-plane axial stresses in the slab are under 0.2 f'c, the effects of out-of-plane buckling and second-order axial force effects on the intensification of out-of-plane moments can be ignored. However, if the stresses exceed this threshold, the buckling load of the slab must be determined.

In calculating the buckling load in intricate conditions of peripheral and intermediate slab supports, 3D modeling and buckling analysis are essential for accurate assessment. In these conditions, the out-of-plane buckling load factor should exceed 2.5, meaning the axial force in the slab should not surpass 0.4 Pcr. This requirement is sometimes sufficient, but that is not always the case. The second-order effects of the axial force must also be evaluated.

When the axial force in the slab is significant, second-order effects can intensify out-of-plane moments. When the structure is still under construction and may not yet have all floor slab loads applied, the intensified moments must be checked. To account for these intensified bending moments, a moment intensification factor should be applied.

In this context, P represents the axial force (axial stress) in the slab, derived from the finite element analysis of the 3D structural model, and Pcr represents the critical buckling force (critical stress) obtained from the buckling analysis of the 3D model for the specific buckling shape of the slab.

4.3.4 Geotechnical Analysis, Design, and Control

4.3.4.1 Suitable and Unsuitable Soil Types for Top-down Construction

In the top-down construction method commonly used in Iran, the process typically begins with the construction of the main columns, and in some cases, support columns within shafts. This is followed by sequential, top-down construction of the retaining walls. In such projects, the local stability of the soil plays a crucial role in determining both the technical and financial feasibility.

Here, the stability of the shaft, followed by the stability of soil segments between columns during sectioning and the construction of the retaining wall, is critical. Therefore, the most suitable soils for this type of top-down construction consist of coarse aggregates combined with high-cohesion fine aggregates. In these soils, the arching effect of coarse aggregates, as well as the cohesion of fine aggregates, contributes to the stability of shaft walls and waiting wells.

Typically, cohesive soils with fine aggregates (such as clay) are particularly suitable for this method, allowing for wall and well excavation without the need for temporary stabilization measures—provided the conditions are assessed and approved by a geotechnical expert. However, if the cohesive content is low and there is a high percentage of clean sand or silt, the soil may be too unstable during shaft drilling or wall excavation. In such cases, temporary stabilization measures, such as using precast concrete lining or shotcreting (see Chapter 3), may be required. Additionally, if the groundwater level is high and the local soil is not compact enough, mechanized shaft excavation using steel casings will be necessary.

4.3.4.2 Geotechnical Limit States

Geotechnical limit state determination methods form the basis of many fundamental equations in geotechnical engineering. These methods underpin calculations related to soil load-bearing capacity, pile load-bearing capacity, soil slope stability, and lateral soil pressure.

In top-down construction, lateral soil pressure is a key factor in designing structural elements and ensuring their stability. The most widely referenced theory for lateral soil pressure in geotechnical engineering was proposed by Coulomb and Rankine. According to their theory, by assuming a failure mechanism behind the retaining walls and the formation of shear bands, the equilibrium of forces and moments can be analyzed, allowing for the determination of lateral forces at the brink of failure.

4.3.4.3 Soil Lateral Pressure

Traditionally, the magnitude and distribution of lateral soil pressure are determined using Coulomb and Rankine theories. Originally developed for unanchored retaining walls, their proposal assumes a triangular pressure distribution. However, in the top-down construction method, due to deformation constraints imposed by floor slabs of underground levels, both the magnitude and distribution of lateral soil pressure may vary.

In top-down excavations, in-situ concrete walls, precast concrete walls, and peripheral columns are typically supported by floor slabs or struts. Under these conditions, soil pressure can fluctuate between active and at-rest states. If support elements (such as concrete slabs and concrete-and-steel beams) lack sufficient stiffness and the wall displacement exceeds 1% of the excavation depth, the soil pressure will approach an active state.

Relying on a triangular pattern can result in underestimating soil pressures at Upper levels and overestimating them at lower levels. Therefore, a lateral soil pressure distribution specific to the top-down method should be used. This distribution can be determined using classical methods for anchored walls or through numerical modeling.

4.3.4.3.1 Classic Lateral Soil Pressure Distribution

Figure 4.1 illustrates the classic soil pressure distribution for different soil types. This distribution is applicable when the ground is level, walls are anchored, the soil has adequate stiffness, and the excavation depth does not exceed one-third of the floor

slab or beam height. If groundwater is present, the pressure distribution changes and must be determined through precise numerical modeling. The classic method can also be applied to excavation depths exceeding 6 meters.

In Figure 4.1, H represents excavation depth, **γ** denotes the soil's unit weight, Ka is the lateral earth pressure coefficient in the active state, and c represents soil cohesion. If the conditions mentioned above are met, the classic method can be used. However, numerical modeling is recommended as well to compare results with the classic distribution. This serves as a validation check for the numerical approach and can aid in feasibility assessments and early construction phase evaluations.

4.3.4.3.2 Lateral Soil Pressure Pattern in Top-down Construction

As previously mentioned, the lateral soil pressure distribution on retaining walls in conventional underground structures is not linear. A linear distribution is observed only when the base of the wall is free to move, which is not the case in top-down construction. In this method, soil pressure at slab-wall connections is typically higher, causing the distribution pattern to deviate from conventional models.

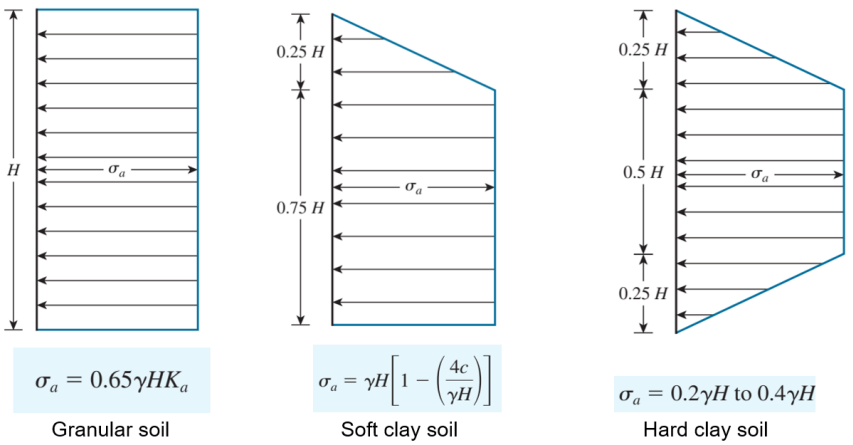


Figure 4.1 Lateral pressure pattern for anchored walls in different soil types

In top-down construction, soil pressure is influenced by several factors, including:

- The type and construction method of the peripheral wall
- The stiffness of the retaining wall
- The arrangement of horizontal elements and floor slabs, including their number and vertical spacing
- The excavation depth at each phase before the installation of beams and floor slabs
- Potential prestress in floor slab elements
- Peripheral wall deformations caused by contraction-induced stress in floor slab concrete
- The geotechnical properties of the soil and its stratification

Lateral pressure on a wall is primarily influenced by three factors: soil pressure, water pressure, and overburden pressure. In top-down construction, lateral water pressure can typically be disregarded, as proper drainage solutions can prevent water accumulation behind the wall.

Overburden pressure is generally caused by two factors: (1) the presence of structures on the ground; and (2) loading pressure from the surrounding environment. While the first factor can be calculated with relative ease, the second can prove more complex. For instance, a minimum of 10 kPa should be considered for secondary streets and 20 kPa for main streets. Additionally, the presence of heavy machinery near excavation walls, along with their operational weights, must be factored in. Overburden pressures should be determined based on soil mechanics principles, ensuring their effects on the wall are properly accounted for in the design.

When designing structural elements in top-down projects, a soil pressure coefficient of 1.6 should be applied. Separating dead and live loads is generally unnecessary unless the consulting engineer determines that a special case requires the superposition method.

In such instances, elastic modeling is necessary to accurately account for overhead loads from dead loads.

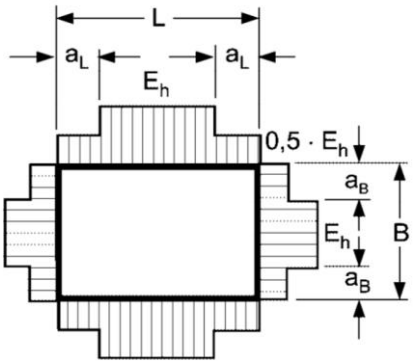
4.3.4.4 Soil Pressure Special Cases

4.3.4.4.1 The Impact of Geometry and Corners

When analyzing excavation stability, plane strain models are commonly used, assuming walls have a significant out-of-plane length. However, in most urban environments, excavation wall lengths are limited, and the excavation footprint has a defined width and length. Additionally, irregularities such as concave or convex corners may be present. These corners create unique boundary conditions that affect the excavation shaft's stress-deformation behavior. The resulting changes in geometry shift the excavation's behavior from a plane strain to a three-dimensional mode to maintain stability. Leveraging the stabilizing effects of concave corners can lead to cost savings.

In dense granular or hard clay soil types, the three-dimensional effects of excavations can be leveraged to enhance stability. These effects result from the boundary conditions imposed by perpendicular walls on either side of the excavation. Coarse-grained soils, in particular, facilitate arching effects, allowing for the efficient transfer of soil forces. However, for this approach to be effective, soil displacement must remain minimal to ensure that the active soil state is properly mobilized.

The three-dimensional effects of excavations can be analyzed using three-dimensional numerical modeling. Additionally, existing analytical relationships can be applied, as demonstrated in Figure 4.2. In this figure, L represents the excavation length, B is the excavation width, and Eh is the calculated lateral pressure at mid-span. Near the corners, at distances of (aL) and (aB), soil pressure can be reduced by 50%. The values of (aL) and (aB) are determined using the following relationships, where H denotes the excavation depth:



$a_L = (0.35 - 0.06 \cdot H : L) \cdot H$ on those sides of length L;
$a_B = (0.35 - 0.06 \cdot H : B) \cdot H$ on those sides of length B.

Figure 4.2 The impact of an excavation's three-dimensional geometry on soil pressure

4.3.4.4.2 The Arching Effect

The stress arching effect can contribute to local stability in shafts during the excavation phase. As such, it can be a valuable advantage in top-down construction by reducing costs. This effect influences both sequential excavations and the construction of peripheral walls. An example is illustrated in Figure 4.3, where the vectors—represented with varying thicknesses—indicate the direction of stress arching within the structural elements of a top-down project. This phenomenon helps stabilize the excavation site, facilitating wall construction. The extent of arching depends on soil conditions and the rigidity of the surrounding structural elements. Its influence should be evaluated based on the engineer's judgment and supported by precise modeling. For temporary stabilization, at least 15% of soil pressure should be accounted for.

4.3.4.5 Numerical Methods of Stress-strain Analysis

One of the most critical stages in the excavation process of top-down projects is calculating the forces acting on structural elements and assessing excavation-induced deformations. This analysis can be conducted using numerical methods with appropriate software solutions. When performing numerical modeling, the following principles should be considered:

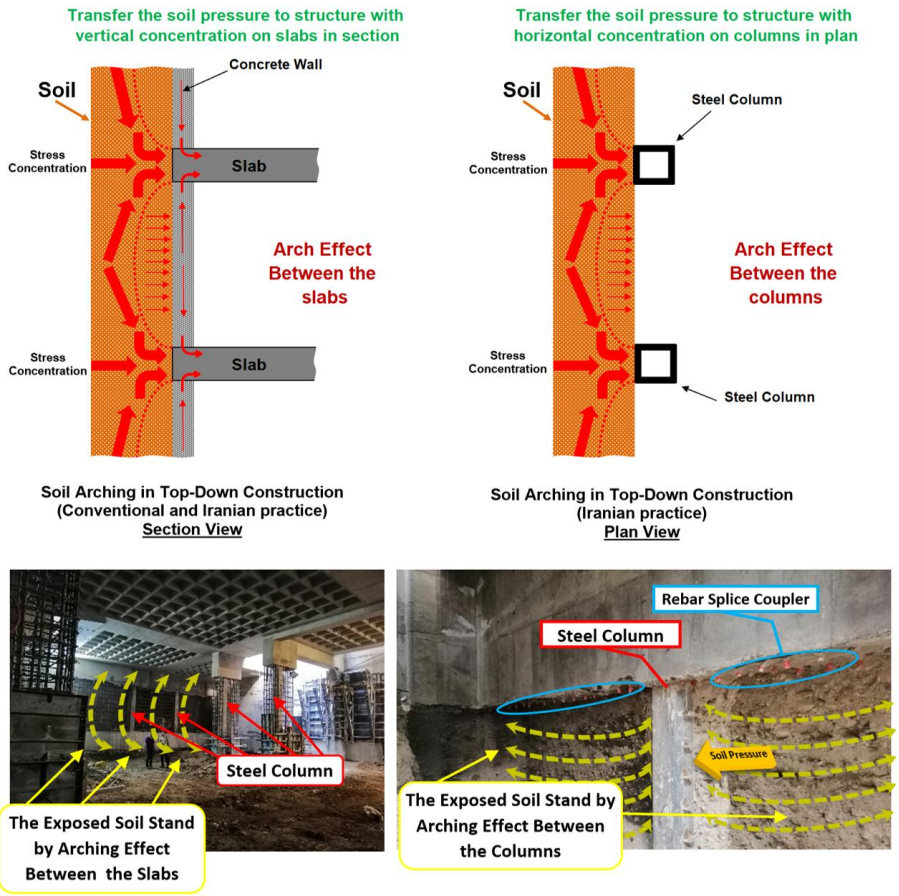


Figure 4.3 Horizontal and vertical arching effect in a top-down project

- A suitable behavioral model should be employed to accurately simulate soil behavior during both the loading and excavation (soil removal) stages. Additionally, the model must account for variations in the soil's modulus of elasticity as depth increases.
- To minimize the influence of boundary conditions, the lateral boundaries should extend at least three times the excavation depth on each side of the shaft. The bottom boundary should be set at a distance of at least twice the greatest plan dimension of the excavation. Figure 4.4

illustrates the general principles for determining boundary distances in excavation modeling.

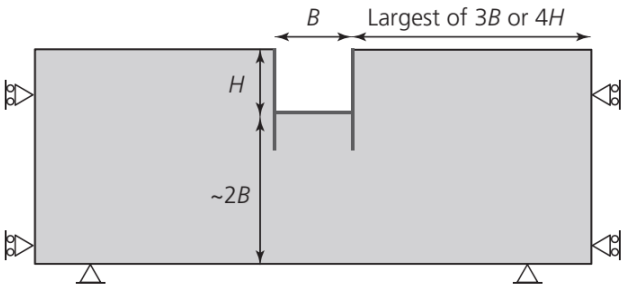


Figure 4.4 Suitable geometry to eliminate boundary condition effect in numerical analysis of excavation

- In a numerical model, the mesh size should be selected appropriately. A sensitivity analysis should be conducted to ensure that the mesh is fine enough so that further refinement does not significantly impact accuracy. It is recommended to use smaller mesh sizes near shaft walls, adjacent building foundations, and other critical points where higher precision is required for deformation analysis.

- All structural elements, including walls, floor slabs, columns, and beams, should be modeled accurately, considering their rigidity. It is recommended to assign elastoplastic properties to structural elements to better represent their behavior. The interaction between structural elements and soil should be well-defined. For elements susceptible to buckling, an appropriate behavior model should be selected in the chosen software to ensure that their maximum load capacity corresponds to their buckling tolerance, preventing any additional load from being applied beyond that point.

- Construction stages, including excavation operations and the installation of structural elements, should be modeled according to actual execution conditions. Stability and excavation-induced deformations should be carefully assessed at each stage.

- Three-dimensional modeling is recommended; however, if appropriate assumptions are made, two-dimensional modeling can be used under plane strain conditions. In 2D models, it is crucial to carefully evaluate beam and column buckling.

- It is recommended to perform numerical simulations of potential in-situ tests, such as PLT and DST, before the primary numerical modeling to calibrate the main model.

- Modeling underground water conditions and drainage during excavation is essential.

- After completing numerical modeling and analysis, the excavation-induced deformations in the surrounding area must be calculated and compared with allowable limits. If deformations exceed allowable thresholds, adjustments should be made by either increasing the stiffness of structural elements or modifying the construction sequence to minimize deformation. This may involve increasing the number of excavation stages, implementing staggered execution, and reducing the excavation depth per stage. Such measures enhance deformation control by allowing more structural elements to be installed progressively.

4.3.4.6 Stability Analysis

At each excavation stage, a thorough stability analysis is essential. Stability analysis principles have been outlined in previous sections. For geotechnical stability, the limit equilibrium method can be applied to calculate safety factors against failure at each stage. This method, commonly used for slope and soil wall

stability assessments, provides a reliable approach to evaluating excavation stability. However, since modeling structural execution can be complex in software based on the limit equilibrium method, an alternative approach—referred to as φ-C reduction—can be used within the finite element method. When adopting this approach, structural elements should be modeled as elastoplastic components. The minimum required safety factor is 1.3 for soil wall stability and 1.5 for overall excavation stability.

4.3.4.7 Observational Method in Design and the Importance of Evaluation During Execution

Top-down projects involve significant geotechnical uncertainties. Therefore, it is essential to monitor excavation-induced deformations and compare them with predicted model values. If discrepancies arise, adjustments to the execution plan and methodology must be made. Additionally, if soil conditions encountered during excavation differ from those identified in geotechnical investigation, the execution plan should be revised, and the stability of structural elements reassessed accordingly. These adaptive measures, collectively known as observation-based design refinement, play a crucial role in geotechnical engineering.

4.3.4.8 Recommended Software Solutions

All verified numerical software solutions can be used to analyze stress-strain deformations in soil. PLAXIS is recommended for finite element analysis due to its user-friendly interface and capabilities in both 2D and 3D modeling. Software solutions based on the finite difference method, such as FLAC, are also suitable. For stability analysis using the limit equilibrium method, Geostudio is a viable option. Additionally, with appropriate adjustments, finite element-based software can also be utilized for limit equilibrium analysis.

Given the variety of software options available, and to ensure reliability, design engineers should model examples from each software library and make sure their results match the outputs provided in the software's documentation. In cases of discrepancy,

both human and systematic errors in the modeling process should be identified and corrected.

Chapter 5

Supplementary Considerations

Chapter 5: Supplementary Considerations

5.1 Introduction

The first two chapters of this guidebook introduced the concept of the top-down construction method, highlighting its significance and feasibility principles. Chapter 3 focused on the execution and intricacies of the construction process. Chapter 4 addressed design considerations, along with geotechnical and structural calculations. This chapter presents additional considerations essential for the effective implementation of the top-down construction method. While some of these points are specific to top-down construction, others are broadly applicable to various construction methodologies.

5.2 The Impact of Deformations Induced by Top-down Construction on Adjacent Buildings

Although the top-down method is generally safe, the excavation process it entails can cause stress fluctuations and deformations in the surrounding areas. Therefore, assessing settlement risks and conducting a thorough risk analysis of nearby structures and urban infrastructure within the excavation's zone of influence are essential.

During the design phase of a top-down project, it is essential to estimate potential settlements in the surrounding areas. This is typically performed as part of the modeling process discussed in Chapter 4. With these estimates in hand and considering allowable ground deformation thresholds, a comprehensive risk assessment can be conducted. Throughout execution, the construction team should carefully monitor and document deformations, comparing their findings to the geotechnical engineer's predictions. Any deviations from expected values must be reported to the

geotechnical engineer so that necessary adjustments can be made accordingly.

If a top-down construction site is located near structures particularly sensitive to settlement—such as historical buildings, bridges, or similar infrastructures—allowable tolerances should be obtained from relevant authorities, including cultural heritage agencies and municipal offices. If the assessed risk exceeds standard thresholds, it is highly recommended to conduct a detailed assessment of structural integrity, particularly regarding foundation stability. This is especially crucial for buildings with deep foundations, where precise modeling of structural elements is necessary to assess potential impacts.

5.2.1 Key Parameters in Assessment of Ground Deformation Impact on Adjacent Buildings

Settlement alone does not necessarily cause damage to a building. To assess its impact, other contributing factors must be taken into account. The following key parameters, depending on the footing type, can influence the extent of potential building damage:

For buildings with isolated footings, damage is primarily caused by differential settlement between footings. The key parameters to consider for this type of structure are the maximum angular distortion (β_{max}) and the maximum vertical settlement beneath the building (S_{max}).

For buildings with raft or mat footings, the key parameter is maximum tensile strain (ε_{max}) which is influenced by the structure's maximum bending (Δ_{max}).

For masonry buildings, two key parameters for evaluating settlement behavior are the building's bending ratio ($DR = \Delta/L$) and the maximum tensile strain (ε_{max}). Since the maximum tensile strain accounts for both the bending and tensile behavior of the structure, it is considered the primary parameter for assessing potential building damage.

Building deformation parameters are represented in Figure 5.1.

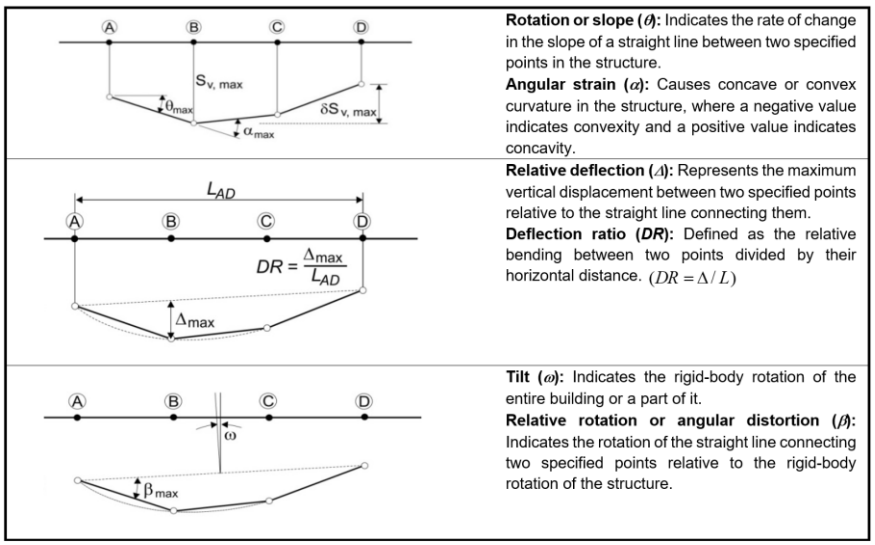


Figure 5.1 Settlement-induced deformation parameters

5.2.2 Predicting Settlement in Areas Surrounding the Excavation

The extent of ground settlement or deformation in areas surrounding a top-down project can be assessed using numerical modeling (finite element or finite difference) as outlined in Chapter 4, or through empirical or analytical methods. According to most references, numerical modeling is recommended primarily when results from these other methods indicate that deformations exceed allowable limits or angular distortions are observed in structures adjacent to the excavation site.

However, the present guide emphasizes that the deformation values (and other critical variables) and settlements caused by top-down construction on the surrounding environment should be determined using numerical methods. Chapter 4 explains the necessity and methodology of geotechnical modeling for a top-down project. For added assurance, it may be necessary not to

model surrounding structures as beams or frames; instead, the free-field displacement of the structures (considering their equivalent loads) should be used. In other words, when using numerical methods to estimate ground displacement, the loads and geometric effects of existing buildings, highways, rail lines, and other infrastructure should be incorporated into the model.

5.2.3 Determining a Building's Vulnerability Index

Typically, damages to surrounding buildings caused by ground deformations can be categorized into four groups:

- Aesthetic damage to the building's facade
- Operational disruptions affecting the structure and its facilities
- Damage to structural elements
- Increased risk to overall structural stability

Damage to surface buildings and structures depends on their condition, structural properties, and the extent of excavation-induced settlement. The first step in assessing building risk is to evaluate their condition and document their characteristics to calculate their Index of Vulnerability (IV) and determine their classification. This process should be carried out in the following stages:

1. Estimating maximum settlement in different areas surrounding the project site and using numerical methods to establish the project's influence zone.
2. Identifying and verifying building boundaries within the influence zone.
3. Creating basic AutoCAD plans based on available plan files, cross-sections, cadastral data, and local planning verifications, along with drafting sketches for each building.

4. Conducting field visits to document building properties, take representative photographs, and complete Building Condition Survey (BCS) forms.
5. Completing IV tables (Tables 5.1 to 5.7 of this guidebook) based on data collected in stages 3 and 4.
6. Classifying buildings based on their vulnerability index.

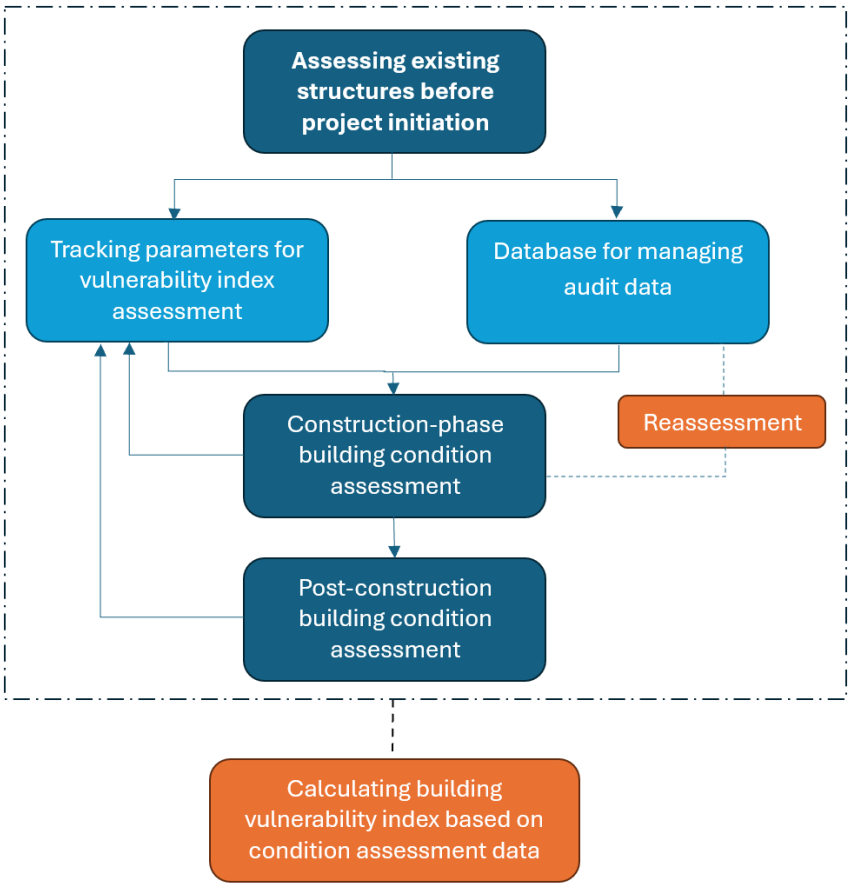


Figure 5.2 Building conditions survey and recording procedure

Table 5.1 Variables Influencing the IV

Variable	Maximum score
Structural behavior	25
Horizontal structural elements	6
Vertical structural elements	6
Foundation - Data collection method	4
Maintenance type (if applicable)	6
Presence of basement levels	3
Building position and orientation relative to the excavation site	25
Orientation	10
The impact of the building group	15
Building's structural performance (functionality)	10
Building's architectural characteristics	20
Historical/cultural heritage	12
Non-load-bearing internal wall type	4
Facade	4
Building's current condition	20
Age	8
Settlement signs in the surrounding area	4
Crack types	8

Table 5.2 IV calculation - structural behavior

IV calculations			
a) Structural behavior Maximum score: 25			
Variable		Index	Value
a.1	Horizontal structural elements		
a.1.1	Wood	6	
a.1.2	Reinforced concrete	0	
a.1.3	Jack arch (brick and steel)	4	
a.1.4	Block-Joist	3	
a.2	Vertical structural elements		
a.2.1	Masonry	6	
a.2.2	Steel	0	
a.2.3	Reinforced concrete	3	
a.2.4	Composite	4	
a.3	Building information source		
a.3.1	Direct (plans, contractors)	2	
a.3.2	Direct (owners, residents, similarity with known buildings)	6	
a.4	Maintenance type (if applicable)		
a.4.1	Unspecified	2	

a.4.2	Increasing the number of openings in the facade or load-bearing walls	6	
a.4.3	Adjustments to maintain the previous structure	0	
a.4.4	Adjustments to improve the previous structure	3	
a.4.5	Retrofitting (load-bearing structure or foundation)	5	
a.4.6	Adding floors	4	
a.4.7	Minor internal adjustments	0	
a.5	Presence of basement levels		
a.5.1	No	0	
a.5.2	Yes	1	
Total		25	

Table 5.3 IV calculation - building orientation

IV calculation					
b) Building orientation and position relative to the excavation site Maximum score: 25					
Variable		Index		Value	
b.1	Orientation	ST	LT	ST	LT
b.1.1	L1/L2 < 0.5	5	10		
b.1.2	0.5 < L1/L2 < 2	6	6		
b.1.3	L1/L2 > 2	10	5		
b.2	The impact of the building group				
b.2.1	Single buildings of the A type: L1, L2 < 2H	15			
b.2.2	Single buildings of the B type: L1, L2 > 2H	5			
b.2.3	Single buildings of the C type: L1 < 2H, L2 > 2H	10			
b.2.4	Single buildings of the D type: L1 > 2H, L2 < 2H	10			
b.2.5	Group of buildings parallel to the trench wall	0	7		
b.2.6	Group of buildings perpendicular to the trench wall	7	0		
b.3	Position relative to the excavation site (values are multiplied by B1, B2)				

b.3.1	X/H < 1	1			
b.3.2	1 < X/H < 2	0.5			
b.3.3	X/H > 2	0			
Total	25				

L1: Average length of the building parallel to the excavation wall
L2: Average length of the building perpendicular to the excavation wall
S.T: Short term
L.T: Long term
x: The distance between the building and the excavation wall axis
H: Excavation depth

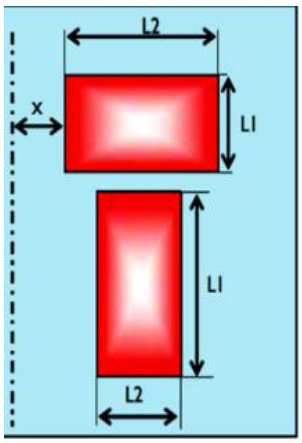


Table 5.4 IV Calculation - Building Performance

IV calculation			
c) Building performance Maximum score: 10			
Variable		Index	Value
c.1	Horizontal structural elements		
c.1.1	Very high priority (hospitals, facilities, structures containing highly sensitive instruments, historical buildings and monuments)	10	
c.1.2	High priority (schools, houses of worship, stadiums, cinemas, shopping centers)	7	
c.1.3	Medium priority (all buildings except for the two above groups, including residential buildings)	5	
c.1.4	Low priority (parking, abandoned buildings)	0	
Total		10	

Table 5.5 IV calculation - Architectural characteristics

IV calculation			
d) Architectural characteristics Maximum score: 20			
Variable		Index	Value
d.1	Historical/Cultural heritage		
d.1.1	No	0	
d.1.2	Yes	12	
d.2	Internal non-load-bearing walls		
d.2.1	Wood	1	
d.2.2	Brick	4	
d.2.3	Prefabricated panels	3	
d.2.4	Aluminum and glass	2	
d.3	Facade		
d.3.1	Artistic or composite tiles (aluminum, glass)	4	
d.3.2	Common (stone, brick)	3	
d.3.3	Plaster (cement)	2	
d.3.4	Other	1	
Total		20	

Table 5.6 IV calculation - Building's current condition

IV calculation			
e) Building's current condition Maximum score: 20			
Variable		Index	Value
e.1	Building's age		
e.1.1	New	0	
e.1.2	Under 10 years	2	
e.1.3	Between 10 to 20 years	3	
e.1.4	Between 20 to 30 years	4	
e.1.5	Between 30 to 40 years	5	
e.16	Over 40 years	8	
e.2	Signs of settlement in the surrounding area		
e.2.1	Yes	4	
e.2.2	No	0	
e.3	Cracks		
e.3.1	Major cracks with extensive networks	8	
e.3.2	Cracks with limited networks	5	
e.3.3	Separate, minor cracks	3	
e.3.4	Cracks on plaster	1	
e.3.5	No cracks	0	
Total		20	

Table 5.7 IV indices - Total

IV calculation		
Variable	Index (maximum)	Value
Behavior	25	
Orientation and position relative to the trench wall	25	
Structural performance	10	
Architectural characteristics	20	
Current condition	20	
Total	100	

The Index of Vulnerability is an inherent characteristic of a building that indicates how far it deviates from ideal conditions. A higher vulnerability index means the building is in worse condition and has a lower tolerance to deformation. The Index of Vulnerability, which reflects the building's degree of vulnerability (or class), is categorized into five ranks, as shown in Table 5.8.

Table 5.8 Buildings' descriptive ranks based on the index of vulnerability (building class)

Index score	0-20	20-40	40-60	60-80	80-100
Index of vulnerability	Negligible	Slight	Low	Medium	High
Building vulnerability (class)	A	B	C	D	E

5.2.4 Building Risk Assessment

Figure 3.5 presents the flowchart for building risk assessment, which includes property inspections, documentation, and surveys. After conducting settlement analysis, defining the top-down excavation site's influence zone, and identifying buildings within this zone, the settlement extent at each building's foundation is calculated. Key variables representing the building's response to predicted deformations (S_{max}, β, ε_h, Δ/l) are then determined. These variables are used to assess potential risk for each building based on the criteria proposed by Burland (1997) and Rankine (1988). The results are combined with the building's Index of Vulnerability (determined through building condition surveys) to assign a risk rank using the risk assessment matrix. It is important to note that no equations are specifically designed for top-down excavation or construction, so the most relevant existing equations should be applied.

Table 5.9 presents the Burland classification system for estimating the impact of underground tunnel excavation on masonry structures (with foundations and tie beams) and structures with surface foundations. This classification is based on the deflection ratio, Δ_{max}/l (Figure 5.4), which is dependent on the maximum tensile strain (ε_{max}). Figure 5.5 illustrates the classification criteria, showing how different horizontal strain values are used to classify damage types. The horizontal strain (ε_h) can be determined using the following equation:

$$\varepsilon_h = \left(\frac{dS_H}{dy}\right) \tag{5.1}$$

Where:

S_H: Horizontal surface displacement
y: Horizontal distance of the point of interest from the excavation axis

The Rankine (1988) classification system applies to structural frameworks with isolated or deep foundations, where the spacing between piles is sufficient to neglect group effects. This system classifies structures based on the maximum settlement (S_{max}) and the maximum angular deformation (β_{max}), as outlined in Table 5.10.

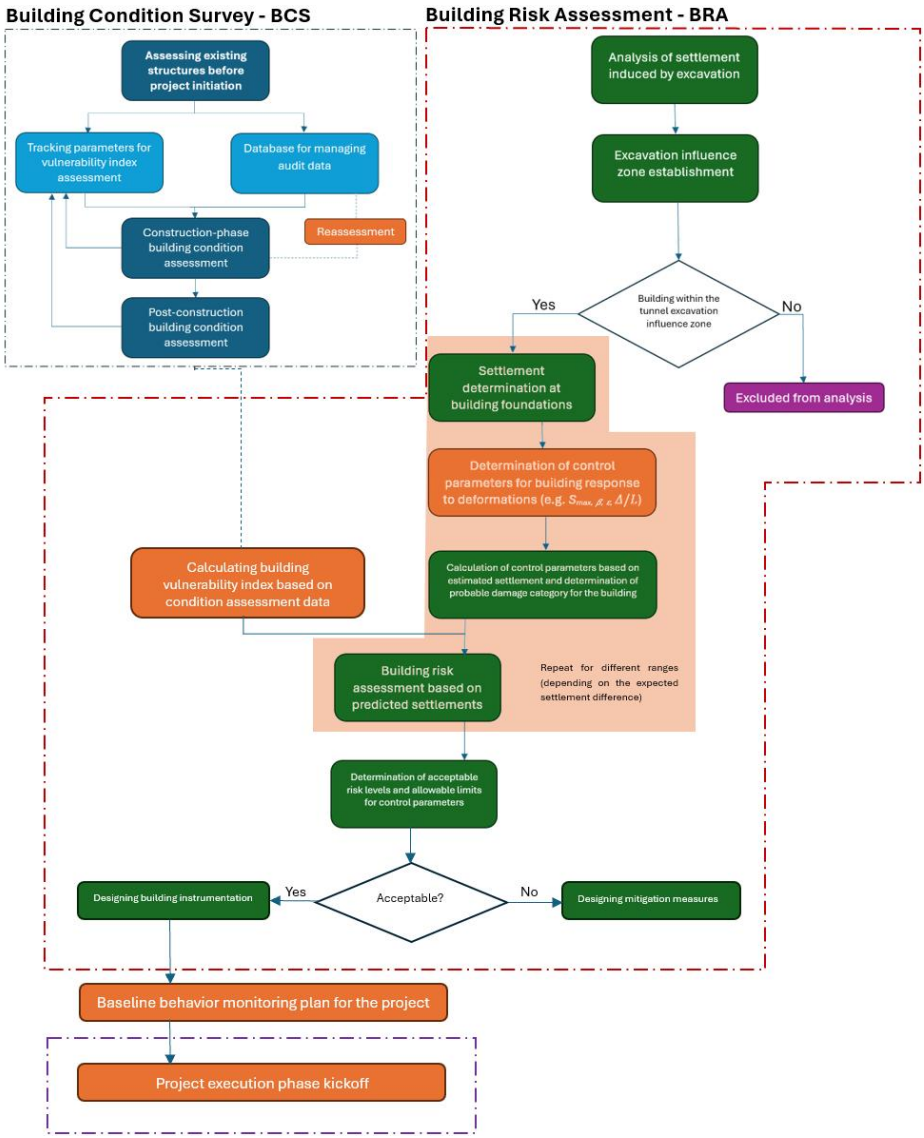


Figure 5.3 The process of conducting inspections, documenting building conditions, and performing risk assessment studies

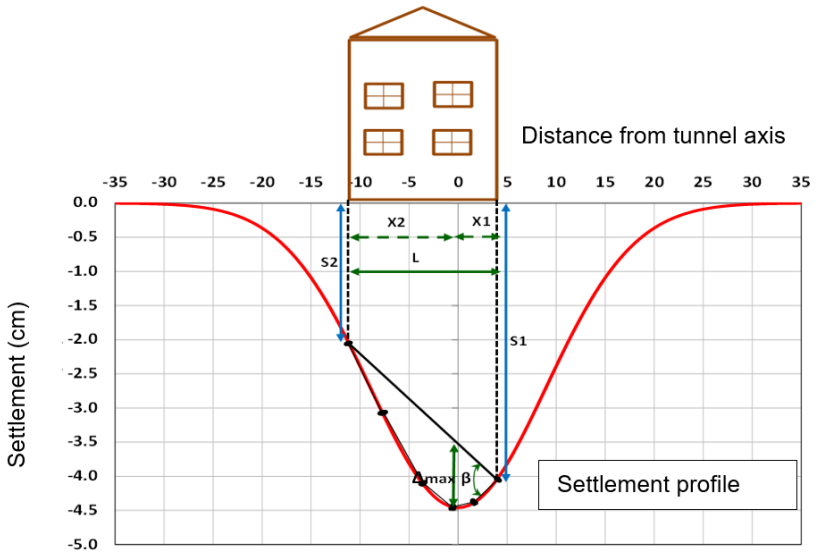


Figure 5.4 Control variables in building assessment using the Burland method

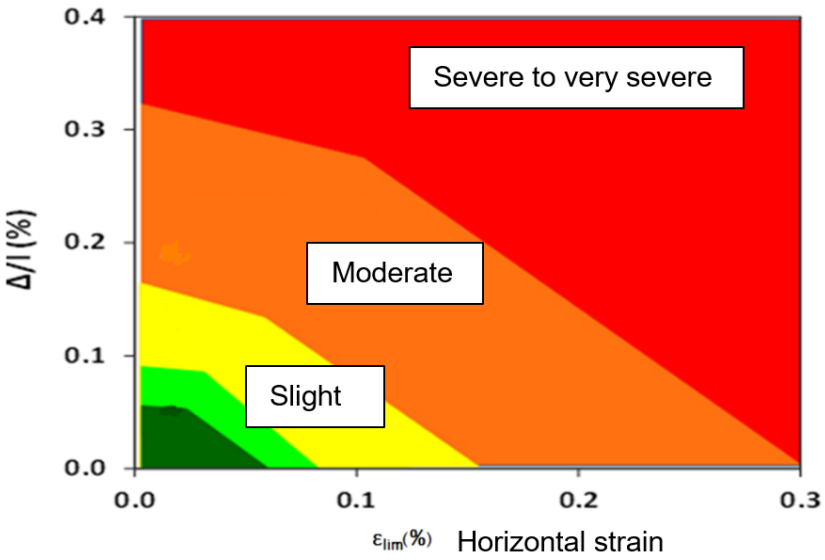


Figure 5.5 Classification of damages based on the ratio of deflection to tensile strain

The connection between the damage category (Index of Vulnerability) and the tensile strain limit (ε_{lim}) in the Rankine system and between the damage category and maximum settlement (S_{max}) and maximum angular deformation (β_{max}) in the Burland system are presented in Table 5.11. The reduction factor of FR should be taken into account when determining the allowable limits of control variables. This factor is designed to apply stricter limits for buildings with a higher vulnerability index. In buildings with higher vulnerability indices, limit values of S_{max}, β_{max}, and ε_{limit} are divided by FR to yield stricter limits (FR is applied to values represented in Table 5.11).

Table 5.9 Damage categories after Burland et al. (1977)

Category of damage	Severity	Description of a typical example	Crack width (mm)	Control variable: tensile strain (% of ε_{lim})
0 (Facade)	Negligible	Hairline cracks	< 0.1	0 - 0.05
1 (Facade)	Very slight	Minor cracks that can be easily repaired during routine decoration. Damage is mostly limited to interior wall finishes, though a thorough inspection may reveal some cracking in external brickwork or masonry.	< 1	0.05 - 0.075
2 (Facade)	Slight	Cracks can be easily filled, although redecoration is likely necessary. Recurrent cracks can be concealed with appropriate coverings. Cracks may be visible on the exterior, and some repointing might be needed to ensure weather resistance.	< 5	0.075 - 0.15
3 (Facade/Functional)	Moderate	The cracks require some widening and can be repaired by a mason. The external brickwork may need repointing, and possibly a small	Typically, 5 - 15, with some exceeding 3 mm.	0.15 - 0.3

		section of brickwork needs to be replaced. Doors and windows may become stuck. Service pipes could crack. Weather resistance may be compromised.		
4 (Functional/ Usability)	Severe	Significant repairs are required, including breaking and replacing sections of walls, especially above doors and windows. The frames of windows and doors are distorted, and the floor shows noticeable sloping. Walls may lean or bulge, and there is some loss of bearing in beams. Service pipes may be damaged.	Typical crack widths range from 15 to 25 mm, depending on the number of cracks.	> 0.3
5 (Structural)	Very severe	Requires extensive repairs, including partial or full reconstruction. Beams lose their load-bearing capacity, walls lean significantly and need support. Windows are broken and distorted. There is a risk of structural instability.	Typically over 25 mm, depending on the number of cracks.	

Table 5.10 Damage categories for frame structures with single foundations after Rankine (1988)

Damage category	Severity	Damage description	Control variables	
			S_{max} (mm)	β_{max}
1 (Facade)	Negligible	No surface damage	< 10	< 1/500
2 (Facade)	Slight	Surface damage without structural implications	10 - 50	1/500 - 1/200
3 (Functional)	Moderate	Surface damage with pipeline damage	50 - 75	1/200 - 1/150
4 (Structural/serviceability)	High	Structural damage with pipeline damage	> 75	> 1/150

Table 5.11 Relationship between the index of vulnerability and damage category

	Index of vulnerability (1)									
Damage category	Negligible		Very slight		Slight		Moderate		Severe	
	0 < IV < 20		20 < IV < 40		40 < IV < 60		60 < IV < 80		80 < IV < 100	
	Reduction factor (F_R)									
	F_R = 1.0		F_R = 1.25		F_R = 1.50		F_R = 1.75		F_R = 2.0	
	Control variable									
	ε_{lim} (%)		ε_{lim} (%)		ε_{lim} (%)		ε_{lim} (%)		ε_{lim} (%)	
	min	max	min	max	min	max	min	max	min	max
0	0.000	0.050	0.000	0.040	0.000	0.033	0.000	0.029	0.000	0.025

1	0.050	0.075	0.040	0.060	0.033	0.050	0.029	0.043	0.025	0.038
2	0.075	0.150	0.060	0.120	0.050	0.100	0.043	0.086	0.038	0.075
3	0.150	0.300	0.120	0.240	0.100	0.200	0.086	0.171	0.075	0.150
4-5	> 0.300		> 0.240		> 0.200		> 0.171		> 0.150	

	Index of vulnerability (2)				
Damage category	Negligible	Very slight	Slight	Moderate	Severe
	0 < IV < 20	20 < IV < 40	40 < IV < 60	60 < IV < 80	80 < IV < 100
	Reduction factor (F_R)				
	F_R = 1.0	F_R = 1.25	F_R = 1.50	F_R = 1.75	F_R = 2.0
	Control variable				

	S_{max} (mm)	β_{max}	S_{max} (mm)	β_{max}	S_{max} (mm)	β_{max}	S_{max} (mm)	β_{max}	S_{max} (mm)	β_{max}
1	< 10	< 1/150	< 8	< 1/625	< 6.7	< 1/750	< 5.7	< 1/875	< 5	< 1/1000
2	10 - 50	1/200 - 1/500	8 - 40	1/250 - 1/625	6.7 - 33	1/300 - 1/750	5.7 - 28.5	1/350 - 1/875	5 - 25	1/400 - 1/1000
3	50 - 75	1/50 - 1/200	40 - 60	1/63 - 1/250	33 - 50	1/75 - 1/300	28.5 - 43	1/88 - 1/350	25 - 37.5	1/100 - 1/400
4	> 75	> 1/50	> 60	> 1/64	> 50	> 1/75	> 43	> 1/88	> 37.5	> 1/100

Based on the type of the structure, first, its index of vulnerability is determined. Then, based on numerical modeling (analytic and empirical methods), control variables (S_{max}, β, ε_h, and Δ/l) are calculated. These values are then used to classify the structure's damage category according to Table 5.12.

Together, these two classification systems form the basis of the structure settlement matrix, which determines the structure's risk class, as outlined in Table 5.12. In this matrix, the potential damage category reflects the likelihood of settlement-related damage based on predicted deformations, while the building class represents the severity of its impact. Each risk category is detailed in Table 5.13.

Table 5.12 Damage categorization matrix based on building class and potential damage

	Damage scale					
Building class	0	1	2	3	4	5
A	I	I	I	II	III	IV
B	I	I	II	III	IV	V
C	I	II	II	III	IV	V
D	I	II	III	IV	V	V
E	I	III	IV	V	V	V

Table 5.13 Risk categories

Risk Category	Description
I. Negligible	Building monitoring during top-down construction
II. Low Risk	Building monitoring during top-down construction
III. Moderate Risk	Mitigation measures should be considered
IV. High Risk	Mitigation measures are required to prevent damage
V. Very High Risk	Damage is unavoidable

5.2.5 Proposed Building Protection Plan

The building protection strategy should be tailored to local conditions and determined by an engineer based on factors such as the predicted risk category, building function, residential status, and other relevant variables. Recommended protective measures for each damage category are outlined in Table 5.14, while Table 5.15 presents an initial action plan and strategy, which should be refined based on the designer's judgment.

Defining allowable settlement values for different structural types can be complex due to the intricate interactions between the structure, foundation (whether shallow or deep), and soil. Neither empirical nor theoretical approaches offer a universal solution. The risk classification guidelines provided here are based on both national and international references and apply only to standard buildings. For special structures, such as historical heritage buildings and bridges near top-down construction sites, allowable settlement limits should be obtained from the relevant authorities or determined through independent, comprehensive studies.

Table 5.14 Protective operations for each building class

Protection class	Type of operation					
	Installing settlement instruments on the building and monitoring	instrument reading at short intervals	Installing specialized instruments	Scaffolding and external protection	Potential evacuation	Improvement
A	✓					
B	✓	✓		✓	✓	
C	✓	✓	✓	✓	✓	
D	✓	✓	✓	✓	✓	✓

Table 5.15 Building Protection Strategies

	Damage Category			Usage					Protection Class			
Risk Class	0, 1, 2	3	4, 5	Historical	Gathering Place (e.g., school, mosque, etc.)	Residential	Commercial Office	Abandoned, uninhabited	A	B	C	D
I, II, III						×	×	×				
				×	×							
IV	×			×	×	×	×					
	×							×				
		×		×	×							
		×				×	×					

		×						×				
			×	×	×	×	×					
			×					×				
V		×		×	×	×	×					
		×						×				
			×	×	×	×	×					
			×					×				

A. Install settlement monitoring instruments on the building and conduct behavior monitoring.

B. Install settlement monitoring instruments on the building and perform frequent behavior monitoring. No preventive measures are required before excavation using the top-down method; however, immediate readiness to address potential risks during excavation is essential. If needed, mitigation measures such as scaffolding, external protection, or even building evacuation should be implemented.

C. Install additional instruments on the building for precise displacement monitoring and conduct frequent behavior monitoring. General building conditions within a 20-meter radius should be inspected 20 days after the excavation face passes. No preventive measures are required before excavation, but immediate readiness to manage potential risks is necessary. If needed, mitigation measures such as scaffolding, external protection, or even building evacuation should be implemented.

D. Implement mitigation and preventive measures (such as injection and ground improvement) before excavation. Install additional instruments on the building for precise displacement monitoring and conduct frequent behavior monitoring. General building conditions within a 20-meter radius should be inspected before the excavation face reaches and again 5 days after it passes. If necessary or economically beneficial, building evacuation may also be considered.

If surveys indicate the need for protection measures of types C and D, it is advisable to validate their effectiveness under actual execution conditions before implementation. Since excavation occurs sequentially and structures are built progressively, it is essential to ensure alignment between survey predictions and real conditions, including construction and excavation sequences.

5.3 The Impact of Excavation-induced Deformations on Urban Facilities in Top-down Construction

Top-down construction generally causes less disruption to urban infrastructure compared to methods like nailing. However, the impact of excavation- and construction-induced deformations on adjacent facilities remains a critical concern even in top-down projects. To assess these effects, observed settlements are compared with allowable limits, which can be obtained from relevant authorities.

The principles outlined in this chapter are largely based on empirical observations and should be regarded as general guidelines rather than strict rules. If these empirical limits are exceeded, factors such as facility type, embedment characteristics, and other executional considerations must be independently evaluated—beyond the standard top-down design—using numerical modeling by expert engineers to determine key parameters like settlement and rotation.
When assessing the impact of settlement on urban facilities, the following controls should be considered:

5.3.1 Comparison of Slopes Induced by Top-Down Structures Adjacent to Urban Facilities with Allowable Empirical Limits

The maximum settlement at the ground surface should be determined through numerical analysis. Empirical studies suggest that in excavations where the central shaft depth is Z0, the influence zone control variable (i) on the ground surface is approximately 50% of the depth in clay soils and 25% in sandy soils. The value of (i) at the depth of urban facilities (Zp) is calculated using the following equation:

$$i = z_0 \left(0.175 + 0.325 \left(1 - \frac{z_p}{z_0}\right)\right) \quad (5.2)$$

Table 5.16 presents empirical allowable slope values at ground level based on maximum excavation-induced settlements.

Table 5.16 Allowable slope values for urban facilities

Pipe type	Allowable limit of the settlement to influence zone ratio (S_{max}/i)
Relatively rigid pipes with a diameter greater than 200 mm	0.012
Relatively flexible pipes with a diameter of less than 200 mm	0.012 - 0.04

5.3.2 Comparing the Probability of Connection Rotation and Pull-out with Allowable Limits

Assuming a conservative maximum rotation in pipe connections, the rotation values for pipes oriented perpendicular or parallel to the excavation site are calculated using the following equations:

$\theta = 2 \ 〖\tan〗^{(-1)} (S_{(v,max)}/\sqrt{2\pi i})$ (5.3)

$\theta = \ 〖\tan〗^{(-1)} (0.4 S_{(v,max)}/i)$ (5.4)

Table 5.17 provides empirical values for connection rotation based on maximum excavation-induced settlement. The pull-out value corresponds to the displacement along the pipe.

Table 5.17 Allowable rotation values in urban facility connections

Pull-out (R) (mm)	Rotation (θ) (°)	Description
-	-	Connections with a history of cracks in lead-yarn joint gas pipes
10	1	Connections in undamaged lead-yarn joint gas pipes
15	1.5	Connections in lead-yarn joint water pipes
25	2.5	Connections in gas or water pipes with rubber gasket joints

5.3.3 Comparing the Stress Generated in Urban Facilities with Allowable Values

Maximum bending and longitudinal strain for pipes crossing transversely through the project section can be determined using the following equations:

$$\frac{d^2 S_v(y=\sqrt{3i})}{dy^2} = 0.446 \frac{S_{v,max}}{i^2} \quad (5.5)$$

$$\frac{dS_h(y=\sqrt{3i})}{dy} = 0.446 + \frac{S_{v,max}}{Z_0 - Z_p} \quad (5.6)$$

Maximum bending and longitudinal strain for pipes running parallel to the project section can be determined using the following equations:

$$\frac{d^2 S_{v(x_f \pm i)}}{dx^2} = 0.242 \frac{S_{v,max}}{i^2} \quad (5.7)$$

$$\frac{dS_{h(x_f i)}}{dx} = 0.242 \frac{S_{v,max}}{Z_0 - Z_p} \quad (5.8)$$

When calculating $\frac{dS_h(y=\sqrt{3i})}{dy}$, a reduction factor ($R_F$) can be used, giving the final value of strain as:

$$\in_{py} = R_F \frac{dS_{hy}}{dy} \quad (5.9)$$

To calculate R_F, the following values are needed:

E_P: Elastic modulus of the pipe material
E_G: Elastic modulus of the soil around the pipe
$A_p = \pi(2r_p - t)t$: Pipe's cross-section (r_p: pipe radius, t: pipe thickness)
$R_a = A_p/\pi r_p^2$: Pipe's area ratio
$K^* = E_p R_a / E_g$: Soil-pipe stiffness coefficient

Figures 5.6 and 5.7 can be used to calculate R_F for a pipe with a diameter of d, with a control variable of i for the excavation influence zone.

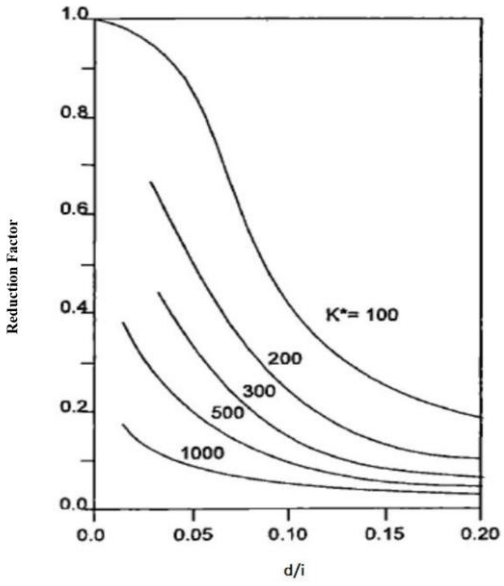


Figure 5.6 Reduction factor for pipes oriented perpendicular to the project (transverse)

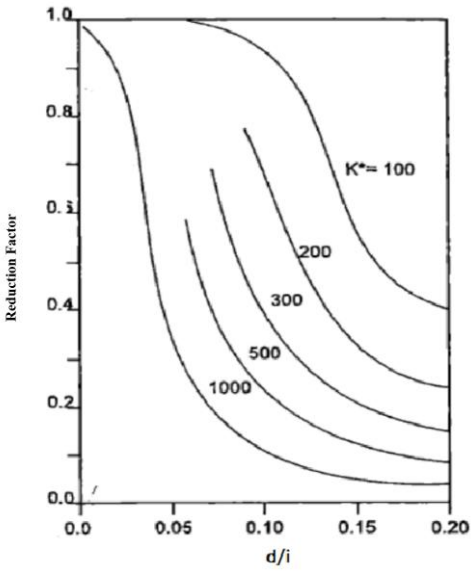


Figure 5.7 Reduction factor for pipes oriented parallel to the project

After calculating the bending and axial strain along with their algebraic sum, the resulting strain values should be compared to allowable limits.

The values in Table 5.18 are conservative. Some references suggest that for pipes made through open or centrifugal casting of gray cast iron with diameters over 300 mm, the allowable strain can be as high as 200 με. For smaller diameters, the recommended limits are 150 με and 100 με. Therefore, the design engineer's judgment, the values in Table 5.18 may be adjusted accordingly.

Table 5.18 Allowable strain values

Material	Allowable strain (με)	
	Compressive strain	Tensile strain
Gray cast iron manufactured through open or centrifugal casting	100	1200
Mild steel	500	700

If urban facility performance is deemed unsatisfactory at any of the three stages mentioned above, the following recommendations should be considered before implementing any measures:

- Establishing allowable values for each stage by consulting relevant authorities
- Utilizing precise numerical models to assess settlement extent
- Incorporating urban facilities into numerical models while adhering to boundary conditions and referencing as-built drawings

If these measures fail to ensure the acceptable performance of urban facilities, the responsibility for identifying retrofitting or improving solutions or relocating the facilities lies with the relevant organizations, while the designer engineer is accountable for implementing measures to mitigate settlement. In other words, any relocation or maintenance of urban facilities must be conducted in full coordination with the responsible organizations.
It is worth noting that the process of detailed modeling of urban utilities falls outside the scope of top-down design studies and should be considered as a separate project.

5.4 Instrumentation and Monitoring

Geodetic and geotechnical monitoring is a widely used approach for mitigating risks and reducing costs in excavation stabilization. They can be used to measure deformations caused by top-down construction. By analyzing monitoring results, safety levels can be determined, and construction stages can be optimized. Given the variability of geotechnical behavior and the complexities of top-down construction, monitoring plays a crucial role in ensuring stability and performance.

This section outlines the primary monitoring mechanisms that can help determine the most suitable approach based on the designer's expertise and the project's significance. However, not all methods discussed here are applicable to every top-down project; they should be considered as recommendations rather than mandatory requirements.

5.4.1 Project Monitoring Objectives

The following are the key objectives engineers aim to achieve when monitoring a top-down project:

- Preventing critical, unforeseen conditions at excavation sites and in surrounding buildings, passages, and surface and subsurface facilities within the project's influence zone, as well as in the building's structure during top-down construction, while ensuring overall safety.
- Monitoring design variables and adjusting the top-down construction process as needed based on the observed results.

5.4.2 Geodetic and Geotechnical Monitoring Responsibility

Geodetic and geotechnical monitoring should be a shared responsibility among relevant authorities to ensure comprehensive data collection and analysis. The details in the monitoring report

are essential components of risk management in top-down projects. Adhering to these principles allows for the early detection and management of unexpected soil behavior, enabling timely corrective and preventive measures.

5.4.3 Key Monitoring Conditions and Criteria

Monitoring is crucial at various stages of a project, including drilling, excavation, and any phase involving soil transformation. The primary focus areas of monitoring include:

- Positions with weak geotechnical conditions
- Locations where underground facilities and passages are vulnerable to settlement
- Sites where adjacent structures fall within the deformation influence zone

Demonstrating the impact of top-down construction on the ground and structures within the influence zone is the primary objective of excavation monitoring. A well-designed monitoring plan should consider all potential risks. Key elements to include in the plan are:

- Wall failures
- Wall deformations
- Settlements or deformations in buildings and surrounding pathways
- Column settlements
- Anticipating potential incidents and implementing preventive measures including:
    - Measurement: Increasing measurement frequency and incorporating additional methods if needed
    - Organization: Streamlining data collection processes and establishing direct communication channels
    - Construction: Adjusting and refining measurement techniques

5.4.4 Monitoring Methods

All collected data and variables during construction, including excavation and structural progress, should be systematically recorded. In addition to general documentation, the following specialized monitoring methods should be considered:

- Tracking deformations in excavation walls
- Monitoring buildings and facilities within the excavation's influence zone
- Measuring surface settlements at benchmark
- Observing inclination in boreholes
- Using multi-point vertical extensometers in boreholes
- Monitoring groundwater levels through observation well

5.4.5 Monitoring Instruments

Excavation projects typically utilize two main categories of monitoring instruments:

- Surface Monitoring Instruments
- Subsurface Monitoring Instruments

Figure 5.8 provides a schematic representation of instrumentation and monitoring techniques used in deep excavations and pile foundations.

Surface monitoring instruments measure settlements and deformations at ground level, providing crucial data on the excavation's impact on surrounding structures and infrastructure. By tracking these deformations, potential risks can be identified and managed effectively.

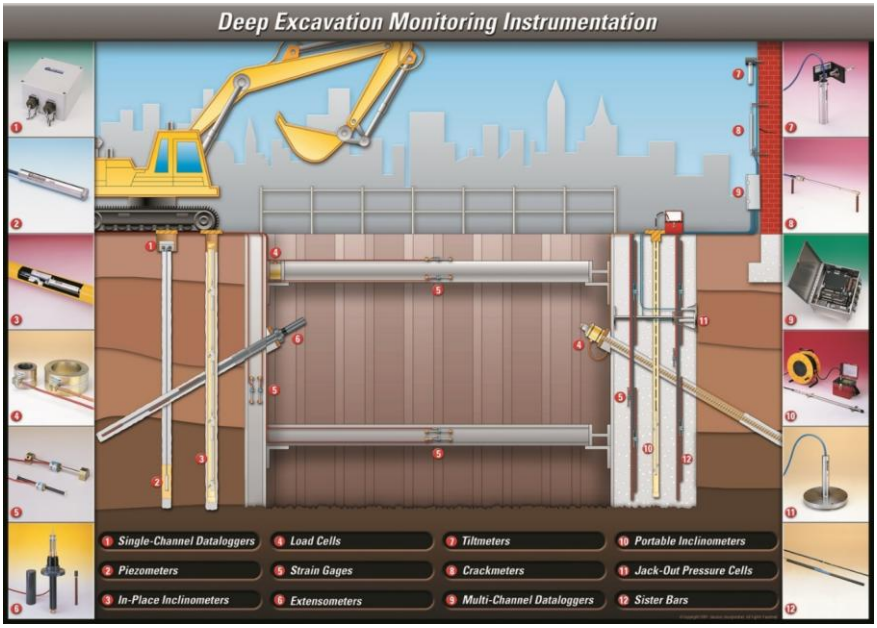


Figure 5.8 Schematic representation of instrumentation and monitoring for deep urban excavations and adjacent structures

5.4.5.1 Surface Monitoring

5.4.5.1.1 Evaluating Ground-level Deformations (Settlement Monitoring)

Although top-down construction offers better control over deformations than other methods, settlement monitoring in top-down projects remains essential. Monitoring points are installed on buildings, infrastructures, streets, and surrounding surfaces, with multiple readings taken relative to a benchmark. The placement of these points should follow an approved design plan. The output of readings is presented as settlement-time graphs. Information related to the progress of various stages of top-down construction is also included in the graphs.

The allowable settlement values for different sections of the project will be determined based on design calculations. The allowable settlement values may change during construction based on monitoring results. Settlement monitoring points are protected

against traffic and potential damage, and if a point is damaged, it is replaced promptly. Once the induced deformations have stabilized, the reference points can be removed.

In the vicinity of buildings or highways adjacent to a project, a more extensive settlement monitoring network is recommended.

5.4.5.1.2 Structural Monitoring

Before construction begins, all buildings and infrastructure within the potential settlement influence zone should be inspected for cracks, joints, and other damages. These inspections should be organized by the client, with priority given to major damages when multiple issues are present. Continuous monitoring is also essential during construction. The initial survey phase involves identifying new damages and assessing existing ones. Crackmeters and jointmeters can be used for monitoring, while remote systems with automatic alarms are recommended for critical structures. Measurement points should be installed on selected structures to monitor leveling at various heights.

Crackmeters and jointmeters provide data in the deformation-over-time format. Additionally, temperature variations and the progress of top-down excavation and construction are recorded. Inclinometer readings output settlement-over-time data and inclination angles, which are derived from differential settlements.

It is important to ensure that allowable values align with design calculations and building conditions. These values may be adjusted during construction based on monitoring results.

All instruments used for monitoring structural behavior must be protected from potential damage and maintained throughout the measurement period. If any instrument malfunctions, it should be replaced immediately. Reference points can be removed once deformations have stabilized.

To assess the impact of top-down construction on buildings within the influence zone, their initial conditions must first be documented. They should then be classified based on their level of vulnerability. Factors contributing to a building's instability generally fall into three categories:

1. Building characteristics
2. Environmental conditions
3. Project-related factors

The following building, environmental, and project-related factors play a significant role in a building's stability:

- Building characteristics: Geometrical dimensions, including length, width, and height (number of floors)
- Side-to-side distance between adjacent buildings and excavation
- Structural type: Steel, concrete, or masonry
- Foundation type: No foundation, isolated, strip, raft, or mat foundation
- Special structural elements: Presence of pools, springs, piles, anchoring, or excavations
- Soil and groundwater conditions: Soil resistance variables and underground water levels
- Water and wastewater systems: Location, orientation, dimensions, and depth of water and wastewater wells, as well as their distance from the project site
- Subsurface infrastructure: Presence of qanats, surface water collection channels, or wastewater redirection systems, including the layout of parallel and perpendicular galleries
- Excavation geometry: Height and shape of the excavation
- Construction sequence: Execution stages of the top-down construction method

5.4.5.1.3 Tiltmeter

To measure and record the degree of misalignment in adjacent buildings, Tiltmeters are installed on exterior walls or concrete slab floors of sensitive structures near an excavation site. Figure 5.9 illustrates a tiltmeter. The precise placement of these instruments requires a site-specific assessment to ensure sufficient security, minimize disruption to residents, and, most importantly, position them optimally from a technical standpoint. Monitoring this variable is especially critical when the excavation level in the top-down method reaches or falls below the foundation level of neighboring buildings before the structure is fully completed.



Figure 5.9 A tiltmeter

5.4.5.1.4 Crackmeter

One of the most commonly used tools for structural monitoring is the crackmeter or jointmeters . These instruments measure the width of cracks and the displacement of construction joints or structural fractures, allowing for the assessment of deformation trends before visible external damage occurs. This enables the assessment of a structure's mechanical behavior during construction. To achieve this, crackmeters capable of readings in one to three axes are installed at various locations. Depending on the function of adjacent buildings and execution constraints, it is advisable to use crackmeters equipped with sensors capable of

providing digital readings from outside the building. Figure 5.10 illustrates a two-dimensional crackmeter with an analog reading gauge. If concerning cracks appear, their progression should be closely monitored.

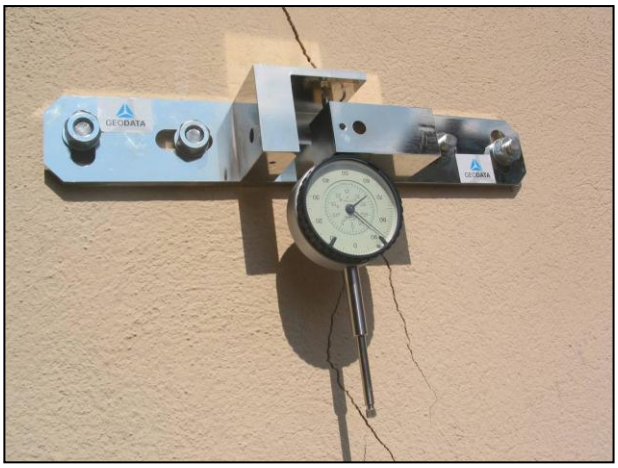


Figure 5.10 A two axes crackmeter with an analog reading gauge

5.4.5.2 Monitoring Environmental Deformations

5.4.5.2.1 Borehole Inclinometer

Inclinometers are used to monitor the horizontal displacement of excavation walls based on design limits, evaluate the performance of piles and retaining structures, and assess the impact of ground movement on adjacent buildings. They also help determine the effects of excavation and top-down construction on adjacent structures.

The inclinometer is an in-borehole instrument designed to measure deformation by detecting deviations perpendicular to the axis of the excavation wall. These deviations are converted into displacements using trigonometric functions. By comparing current and initial readings, changes in lateral deviation can be identified, revealing ground movements. The cumulative changes from each measurement cycle are plotted to generate a high-

precision displacement profile, which helps determine the magnitude, depth, direction, and rate of displacement.

Boreholes should be drilled outside the top-down construction zone, near peripheral walls of the project. In urban areas, this may require access to neighboring properties, which could involve legal constraints. Therefore, drilling feasibility must be evaluated beforehand based on the conditions of surrounding structures. The boreholes should be positioned approximately 1 meter from an excavation wall, with a depth extending 4.5 to 5 meters below the bottom of the excavation. The exact coordinates will be determined in later design stages. Initial inclinometer readings must be taken before excavation begins and then after each construction phase until deformations become negligible. The use of this instrumentation in top-down construction is particularly recommended when important adjacent structures may be affected.

5.4.5.2.2 Single- and Multi-point Borehole Extensometer

To measure soil displacements caused by top-down construction, borehole extensometers are used (Figure 5.11). These instruments consist of one or more anchor points embedded at various depths within the borehole, along with a reference point at the borehole collar. They typically feature steel rods, synthetic fibers, or connecting wires that link the anchor points to the reference point. Borehole extensometers allow for measuring the relative displacement between the anchor points and the borehole collar, as well as the displacement between any two anchor points.

To determine absolute displacement, at least one measurement point must be positioned in an area unaffected by movement or be measurable using a reference point through surveying methods. If surveying methods are used, the accuracy of the absolute displacement data will depend on the precision of the surveying technique.

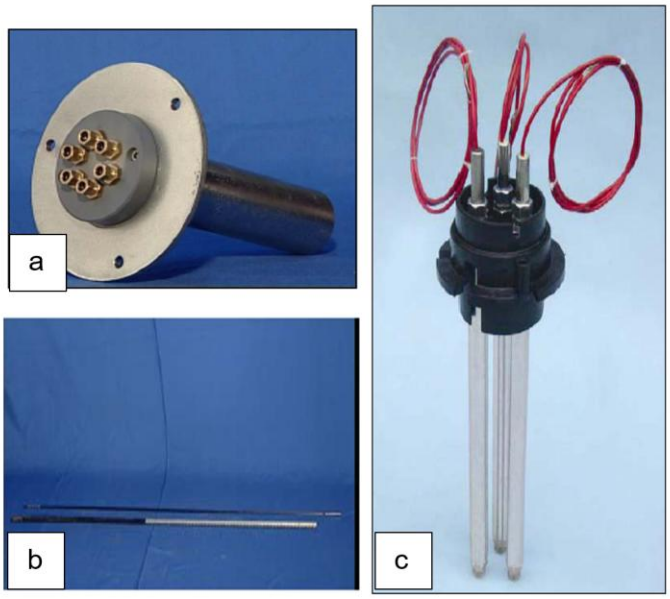


Figure 5.11 Components of an extensometer: a) an extensometer head, b) an end anchor rod and extension rod, and c) an extensometer head with a transducer

This instrument is widely used in tunneling. In the context of top-down construction in Iran, however, we have yet to encounter any reports of its application. In urban excavations and top-down construction—often conducted in densely built areas—deploying this instrument typically requires drilling boreholes within neighboring properties, which can pose legal challenges. Therefore, its use in top-down construction is recommended only for highly specialized projects.

5.4.5.2.3 Observation Well

Groundwater levels can be monitored by measuring pore water pressure using piezometers. Additionally, water levels in existing wells can provide valuable data. The required reading precision is 2 mm, with a maximum allowable error of 5 mm. Wells must be protected from potential damage and maintained throughout the monitoring period. In most cases, once deformations have stabilized, observation wells can be filled in. The significance of groundwater level monitoring depends on whether hydrostatic

pressure was considered during the top-down structural design phase.

5.4.5.2.4 Within-pit Monitoring (Top-down Construction Zone)

Internal project monitoring methods include periodic inspections of various construction stages, geological surveys, and deformation monitoring of excavation walls using geodetic instruments. Additionally, compressive and tensile stresses in peripheral walls are measured using strain gauges embedded in concrete.

The output will be presented as deformation over time and deformation vectors. Excavation progress must also be recorded and reported. Design limits, based on calculations and structural conditions, should be adhered to, though they may be adjusted during construction based on monitoring results.

Monitoring points must remain intact throughout all stages, and any damaged points should be replaced immediately. They can be removed once deformations stabilize.

During excavation, readings may need to be taken every six hours. For critical points and deformations, finite element analyses are used for comparison.

If readings exceed critical thresholds, excavation will be halted, structural elements will be constructed more rapidly, and, if necessary, supplementary stabilization systems or revisions to the top-down design and construction sequence will be considered.

5.4.5.2.5 Concrete-embedded Strain Gauge

Strain gauges embedded in concrete structural elements, such as peripheral walls, piles, or other restraining components, can serve as alternatives to pressure cells or be used alongside them to validate readings. These gauges indirectly measure stress fluctuations, whether compressive or tensile. Unlike pressure cells,

strain gauges can also detect tensile strains, making them particularly useful. However, when using strain gauges, the concrete's stress-strain properties must be carefully determined. An example of concrete-embedded strain gauges can be seen in Figure 5.12. In top-down construction, these gauges are commonly used in peripheral concrete walls subjected to soil pressure and in concrete columns.

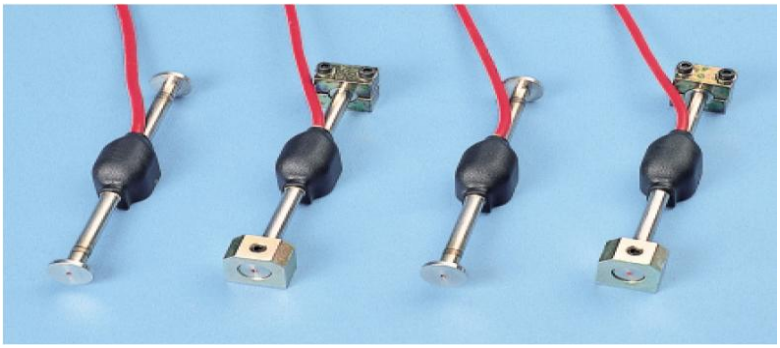


Figure 5.12 An example of concrete-embedded strain gauges used for geotechnical monitoring

5.4.6 Alert System Based on Monitoring Results

Top-down construction projects face geotechnical risks similar to those in other excavation projects. Monitoring and behavior analysis are essential for managing these risks. To effectively address them, different action levels must be established based on monitoring results. Accordingly, various alert levels are defined, considering settlement around the excavation and deformations in the excavation walls.

5.4.6.1 Alert System Based on Settlement Around Excavation

The alert system defines three levels based on settlement severity. For surface settlement, three key variables are considered: predicted settlements due to excavation, allowable settlements for buildings and infrastructure, and the acceptable tolerances and extent of settlements.

5.4.6.1.1 Settlement Awareness Level

The Awareness Level is defined as 85% of the predicted surface settlements and 70% of the allowable settlement for buildings and infrastructure. When readings reach this level, the contractor's geotechnical expert must notify the engineering team and submit a report to the site supervisor or relevant authorities. This report should analyze ground conditions, identify the factors contributing to the Awareness Level, and propose risk mitigation measures. Additionally, geotechnical monitoring should be adjusted accordingly, with an increased frequency of readings.

5.4.6.1.2 Settlement Alert Level

The Alert Level is defined as 100% of the predicted surface settlements and 90% of the allowable settlement for buildings and infrastructure. When readings reach this level, the contractor's geotechnical expert must, as with the Awareness Level, notify the engineering team and prepare a report for the relevant stakeholders. Following the report, a site visit by the consultant and contractor is arranged to assess potential damages and any other contributing factors. The findings from this visit must be documented in a report. Geotechnical monitoring should be adjusted accordingly, with an increased frequency of readings. All data must be communicated to the consultant, designer, and senior project engineers within six hours of the readings being taken.

If the on-site movements cannot be controlled by the contractor, the engineers, and designers must assess potential scenarios and determine necessary design modifications. The contractor must also be prepared to implement these modifications as required.

If design modifications are necessary to avoid reaching the Danger Level, engineers, designers, and the contractor must work together to implement measures that minimize deformations and prevent further movement. The client must also be informed of current conditions.

5.4.6.1.3 Settlement Danger Level

The Danger Level is defined as surface settlements exceeding 120% of the predicted values and 100% of the allowable settlement values for buildings and infrastructure. At this stage, immediate design modifications must be implemented. If necessary, evacuations should be carried out, streets closed, and utility lines and infrastructure services shut off to prevent damage and casualties. Additionally, site inspections must be conducted to assess damage to concrete linings, residences, infrastructure, and all other structures within the project's vicinity.

Geotechnical monitoring should be adapted to prevailing site conditions, with reading frequencies increased to a minimum of twice per day. All data must be shared with the consultant, designer, and project authorities within six hours of recording. The client must also be kept informed of the situation at all times.

5.4.7 Minimum Instrumentation Requirements for Top-down Construction Projects

The essential instrumentation for monitoring a top-down construction project includes a total station theodolite and geodetic targets. If specific conditions arise, additional instruments—such as extensometers, inclinometers, strain gauges, crackmeters, tiltmeters, pre-settlement gauges, leveling cameras, load cells, and piezometers—may be used based on the supervision team's recommendation or the client's request.

5.5 Surveying Considerations

The accurate implementation of design plans is the most critical aspect of any civil engineering project, as errors in execution can often be irreversible. This section covers key techniques for layout and implementation in the top-down construction method, along with structural monitoring methods based on surveying techniques. It is essential to strictly adhere to general surveying principles with heightened precision at every stage of the project.

5.5.1 Setting Out Points

Beyond general surveying considerations, the most critical aspect of surveying in the top-down construction method is the precise setting out of points at the bottom of the shaft and the accurate installation of columns. This includes ensuring the correct horizontal positioning (x, y) and vertical elevation (z) of the columns. The following recommendations are essential for ensuring the accuracy of this process:

- To set out the horizontal position of the columns within the shaft, a base template must first be constructed and securely installed at the top of the shaft. Plumb bobs are then used to transfer the four corner points of this template to the bottom of the shaft. For enhanced accuracy, it is recommended to use an additional column template inside the shaft, which should be positioned and secured using plumb bobs. Since plumb bobs may oscillate, using four simultaneously—one at each corner—helps ensure greater precision in point transfer.
- If a continuous gallery connects the shafts, using a string line within the gallery to transfer points and set out axes is an effective method for minimizing errors in point transfer.
- Setting and controlling the column elevation during installation is done from the column head, which generally presents no particular challenges. However, it is essential to ensure that precise markings are made on the column head to serve as reference points for level control. Additionally, it is recommended that connections on steel columns be installed separately rather than directly on the column itself to minimize sensitivity to elevation codes. This approach simplifies and speeds up the installation process.

- For more precise and trouble-free installation of columns, especially steel columns, it is recommended to stretch a string line along all four sides of the column before lifting it. This helps detect any potential curvature or deformation that may have occurred during construction, ensuring the column is straight and free from warping.

5.5.2 Deformation Monitoring

Monitoring, displacement measurement, and deformation control can be conducted for both surrounding structures and excavation walls. This process involves installing reflectors at designated reference points on the structure. Using a geodetic network designed around the site, readings are then taken from predefined stations. Typically, a high-precision local geodetic network is established to create control points, detect deformations, and perform precise measurements.

In any geodetic network, different approaches can be adopted based on project objectives. The goal of network design—carried out by surveying specialists—is to determine the most suitable method for achieving these objectives. Key factors in designing a control network include precision, reliability, and expense.

The establishment of a geodetic network follows a structured process: first, the design phase defines observation types, accuracy requirements, and necessary instruments. Next, control points are created. Finally, measurements are carried out.

Given the high precision required in geodetic networks, achieving accurate coordinate measurements demands high-precision observations. The network design must account for the impact of observational errors on point coordinates. Additionally, when setting precision criteria, it is assumed that only random errors affect the observations, with systematic errors minimized or eliminated. Furthermore, the network must exhibit both internal and external reliability. Various approaches can be taken in

designing a geodetic network, and the selection of the most appropriate method is determined by surveying specialists.

5.6 Safety Considerations in Top-down Construction

Safety considerations for top-down construction and excavation practices have been discussed throughout this guide. In essence, all topics covered relate to the design, implementation, monitoring, and instrumentation necessary for safe excavation. However, this section focuses specifically on worker safety and labor laws.

The hazards present in top-down construction sites are not unique and are commonly found in conventional construction projects. However, their likelihood may vary depending on the nature of the work.

To avoid unnecessary repetition, general safety guidelines are not included in this guide. Readers should instead refer to relevant sources. Since these hazards are well-documented, various approaches to and solutions for mitigating them are outlined in existing regulations and standards.

Key resources include Chapter 12 of the Iranian National Building Codes, *Safety and Protection of Workers During Construction*, as well as the technical protection and occupational health regulations published by the Ministry of Cooperatives, Labor, and Social Welfare. These include *Safety Regulations for Construction Sites, Manual Well Digging, Working at Heights, Personal Protection, and Tunnel Construction*.

Given the increased likelihood of certain hazards in the top-down construction method, this section highlights and introduces critical risks, with some discussed in greater detail (Table 5.19), ensuring that necessary preventive measures align with existing regulations.

From a project management perspective, a well-defined Health, Safety, and Environment (HSE) management plan is essential. This ensures proper implementation, monitoring, and inspection of safety measures, tailored to the specific needs of each project and organization.

In addition to complying with the referenced safety guidelines and regulations, as well as the considerations discussed in this guide, it is crucial to emphasize that during any operation other than shaft excavation—such as concreting or steel column installation—the presence of personnel inside the same shaft is strictly prohibited.

If access to the work area is necessary for technical controls, movement should occur through another shaft, with entry to the designated location provided via a gallery. If no gallery has been excavated, alternative safety measures must be implemented. These include creating a properly sized refuge niche in the shaft wall, ensuring adequate air supply at the designated depth, and restricting movement through the shaft to non-hazardous times.

5.6.1 Falling into Shafts/Wells or Openings

One of the key characteristics of the top-down construction method, particularly the SPW approach, is the presence of numerous shafts and wells across the site, especially during the initial stages when the construction site is not yet fully established. These shafts, often widespread and relatively large, pose significant safety hazards, particularly due to the concurrent execution of other activities alongside shaft excavation.

Table 5.19 Potential and notable hazards in the top-down construction

Potential Hazard
Falling into shafts or wells
Sinking into undiscovered underground voids, such as qanats and sewage chambers, during underground excavation
Drowning due to sudden influx of water into the shaft
Risk of electric shock when using electric demolition hammers in wet shafts
Insufficient air supply in underground voids during excavation or installation of structural components, or the presence of harmful gases or vapors during activities such as concreting
Falling objects (light or heavy, including structural components) into the shaft during excavation or installation
Collapse of the shaft opening during installation of components caused by the accumulation of personnel or other loads
Falling from heights in partially completed structures
Collision of excavation machinery with structural components, other instruments, and personnel due to the limited working space available under floor slabs
Collapse of exposed, unprotected soil walls during excavation
Hazards caused by insufficient lighting and visibility in underground spaces
Hazards caused by reduced human alertness due to insufficient oxygen and fresh air supply in underground spaces
Hazards caused by structural instabilities during sequential constructions

To mitigate the risk of falls, it is essential to install sturdy protective barriers or guardrails around each shaft during excavation and to securely cover completed shafts. In areas with loose or unstable soil at the upper levels, applying a sprayed concrete layer (shotcreting) or installing tubing should be considered to prevent conical failure at the shaft head. Additionally, ensuring adequate lighting at night and covering shaft openings during site shutdowns are simple yet effective measures to enhance safety.

Furthermore, given the increased number of openings and voids in structures built using the top-down method—such as soil discharge openings—it is strongly recommended to install appropriate guardrails around all such openings to prevent accidents.

5.6.2 Sinking into Undiscovered Underground Voids

Underground work, particularly in urban areas with hidden cavities, always carries the risk of encountering unexpected voids. During shaft or well excavation, it is not uncommon to come across old sewage storage tanks, abandoned water reservoirs, qanats, underground channels, or similar structures. This risk is heightened in top-down construction, where excavation is often performed manually due to space constraints or because of the shaft's geometry and dimensions.

The danger becomes even more severe if these hidden voids contain water or sewage, which entails drowning risk. Therefore, conducting thorough field investigations is essential. This includes reviewing qanat maps, examining sewage systems of demolished as well as operational neighboring buildings, identifying potential underground facilities, and assessing the project area's subsurface conditions.

In addition, excavation workers must receive specialized training on identifying and responding to potential hazards and an alert

assistant should always be stationed at the top of the shaft. Strict enforcement of safety measures, particularly the continuous use of a safety harness securely anchored outside the shaft, is critical to ensure worker safety.

5.6.3 Drowning Due to Sudden Influx of Water into the Shaft

In water-saturated areas, there is a significant risk of a shaft unexpectedly encountering a large volume of water from a qanat or sewage system. In such cases, the danger of drowning is real and must be taken seriously.

To mitigate this risk, several precautionary measures are essential: conducting a thorough investigation of the area's hydrogeological conditions, training excavators on how to respond to sudden water ingress, ensuring proper lighting in the shaft for clear visibility from above, and requiring an alert assistant to continuously monitor activities from the top of the shaft. Additionally, workers should be trained to recognize unusual sounds that may indicate water infiltration and the use of a securely anchored safety harness must be strictly enforced at all times.

5.6.4 Risk of Electric Shock in Wet Conditions

In underground working conditions, the use of electrical devices and tools around wet soil is restricted due to the risk of electric shock. Specifically, electric breakers are prohibited in wet soil. Pneumatic (air-powered) hammers must be used instead. For other instruments, similar precautions must be taken. Workers must wear properly insulated gloves and footwear to minimize the risk of electrocution.

5.6.5 Underground Fresh Air Supply

In underground conditions, inadequate ventilation can lead to oxygen deficiency, reduced alertness, and serious risks to workers. This issue becomes more critical with increasing depth and is further exacerbated by the presence of gases from sewage or other sources. Traditionally, experienced excavators manage fresh air

supply in shafts based on their expertise, but strict adherence to safety regulations and standards for manual shaft excavation remains essential.

However, in other underground scenarios during top-down construction—where machinery such as excavators and loaders operates below ground level, consuming oxygen and generating pollutants—ventilation challenges must be addressed through engineered solutions. This topic is discussed in detail in Section 3.4.2 of the present guide.

5.6.6 Objects Falling into Shafts or Openings

In top-down construction, some operations take place inside shafts, and a significant portion of operations occurs on the lower levels of the structure. Due to the high volume of traffic and simultaneous activities at ground or street level, the risk of objects falling into shafts or through openings onto workers below is considerable.

To mitigate this hazard, it is essential to consistently reinforce safety protocols and ensure all workers are aware of the potential presence of personnel near openings. Regularly clearing the worksite of debris, stones, and other waste materials, as well as implementing raised edges around shafts and openings to prevent accidental falls, are effective measures in reducing risks.

5.6.7 Shaft Opening Collapse

During certain stages of construction, heavy loads may be applied to the shaft or well opening. The installation and splicing of steel or concrete columns or the suspension of columns within the shaft are some of the instances where such heavy loads may be applied. In many cases, the shallow soil layers at a shaft or well opening may consist of backfill, loose soil, or construction debris, making them susceptible to failure under heavy loads.

This issue not only disrupts construction but also poses significant safety risks, especially since many workers are typically present

around the shaft during these operations. Therefore, a specialist must carefully assess conditions, and if necessary, reinforce the shaft opening. Suitable solutions include tubing, installing steel liners, or creating a reinforced concrete cover using shotcrete or sprayed concrete.

5.6.8 Collision of Excavation Machinery with Personnel or Structural Components

Working underground is an inherent aspect of top-down construction, presenting several challenges such as confined spaces between columns and structural elements, limited lighting, and working under a floor slab. These factors significantly increase the risk of collisions between excavation machinery, workers, and structural components, particularly when multiple work fronts operate in close proximity. Collisions with workers can be fatal, and structural element collisions can cause severe damage. To mitigate these risks, the following precautions must be carefully observed:

- Ensuring adequate lighting is provided in accordance with the technical guidelines outlined in Section 3.4.1 of this guidebook.
- Regularly cleaning and maintaining lighting fixtures to remove dust and ensure proper functionality.
- Employing skilled and experienced operators for underground machinery and providing necessary training.
- Equipping all machinery with appropriate warning lights and horns.
- Requiring all workers in underground areas to wear high-visibility reflective vests.
- Installing impact buffers and reflective strips on structural components, such as columns and exposed roof edges, to reduce collision risks.
- Organizing work fronts strategically to minimize hazards as much as possible.

5.7 Legal and Contractual Considerations in Top-down Construction

A project using the top-down construction method remains entirely within its property boundaries, eliminating the need to encroach on adjacent properties. This is a significant advantage from both legal and land acquisition perspectives, making the top-down approach the preferred stabilization option in many cases.

No special legal permits or approvals are required for the implementation of the top-down method. However, it is generally advisable to document the existing condition of neighboring buildings before construction begins. This can be done by obtaining an official assessment from authorized entities and certified experts appointed by the judiciary. Taking this precaution helps manage and mitigate potential claims from adjacent property owners regarding construction-related damages.

Another critical legal aspect of the top-down construction method relates to the contracts governing its implementation. Due to the method's numerous technical and geotechnical interactions, its implementation can be uncertain and largely based on observational design principles. This results in frequent modifications and a high likelihood of changes throughout the project. Additionally, the complexity and overlap of activities in this method are significantly greater compared to conventional construction. As a result, contracts for top-down projects require carefully drafted legal provisions to address two key aspects:

a) Managing Claims During the Project: The dynamic nature of this method often leads to claims arising from design adjustments and changes made to the construction process. Contracts must include mechanisms for efficiently handling and resolving such claims.

b) Defining Responsibilities and Accountability: Given the high degree of overlap between processes, maintaining clear lines of responsibility among project stakeholders is

essential to prevent disputes and ensure smooth coordination.

To address these challenges, contracts should establish a comprehensive legal framework that accounts for the method's unique demands, providing clarity, fairness, and accountability throughout the project's execution period.
Due to the inherent unpredictability of top-down construction projects, the rate of variations is high, often resulting in claims from various stakeholders. These claims can take different forms, such as financial claims from the contractor or disputes over scope and quality raised by the employer. Furthermore, given the complex interactions between activities, determining responsibility for a particular event or unfavorable outcome is often challenging.

The following are recommendations for creating more effective conditions for managing claims and enhancing contractual frameworks to better accommodate the unique demands of this construction method.

It is highly recommended to avoid dividing top-down construction projects among multiple contractors, a practice commonly referred to as "subcontracting." While it may appear that assigning tasks such as shaft excavation, column installation, and structural construction to different contractors could reduce costs compared to awarding the entire project to a single contractor, this approach often leads to complications. If issues arise, the fragmented distribution of responsibilities can result in quality degradation, increased costs, and other challenges. Identifying the accountable party becomes particularly difficult, as the overlap of activities obscures clear lines of responsibility. Conversely, consolidating responsibility under a single main contractor allows for better control over project deliverables and accountability.
For the same reason, separating design from execution in this method is not advisable. Instead, design-build contracts are strongly recommended to ensure seamless integration between

design and construction, reducing conflicts and improving project efficiency.

Given the high likelihood of changes in top-down construction projects, contract structures must be flexible enough to accommodate adjustments. Lump-sum contracts are generally unsuitable, as they either increase financial risk for the contractor or inflate costs for the employer. In contrast, reimbursable contracts offer a more adaptable approach.

Among these, management contracts—such as cost-plus fixed fee (CPFF), cost-plus incentive fee (CPIF), and cost-plus award fee (CPAF) agreements—are particularly noteworthy. These contracts reimburse project costs while providing additional compensation as either a fixed amount or a percentage of costs. They may also include incentive mechanisms, such as bonuses for cost savings or accelerated timelines, to align the interests of both parties. Properly structuring these agreements can optimize shared benefits for both the employer and the contractor. Conversely, unit-price or schedule-of-rates contracts pose a high risk of variations and potential claims, making them less suitable for the complexities of top-down construction projects.

5.8 Execution Management in Top-down Construction

Top-down construction is an underground building method that shares many similarities with tunneling operations. In tunneling, the success of a project heavily depends on designing an efficient execution system and selecting the most suitable construction method. Likewise, in top-down construction, achieving optimal cost and time efficiency is directly tied to the effectiveness of the chosen execution strategy. Therefore, successful management of top-down projects requires careful planning and a sharp focus on execution efficiency.

In top-down construction, all soil within the project area must be excavated and removed through one or two designated openings.

Placing these openings near the project's centers of activity minimizes the need for extensive underground transportation of machinery, thereby accelerating progress and reducing costs.

A critical aspect of this method is the process of lifting excavated soil to the ground level. The choice of machinery for this task directly impacts both the location of the openings and the overall efficiency of operations. An optimal execution strategy must consider multiple variables, many of which can be quantified and formulated. In tunneling engineering, time and cost functions are often developed for different scenarios, allowing for an optimized operational plan. Similarly, in top-down construction, analyzing and optimizing these parameters ensures a more efficient and cost-effective approach.

Another defining characteristic of the top-down construction method is the overlap of operational activities. For example, reinforcement work and concrete pouring often take place simultaneously with excavation. Designing an execution plan that minimizes these overlaps is essential for efficiency and safety. This need becomes even more critical when superstructure and substructure levels are being built concurrently, requiring a highly coordinated and systematic approach. Chapter 3 of this guide provides a detailed discussion of this topic.

5.9 Value Engineering in Top-down Construction

Value engineering is a structured approach aimed at evaluating and optimizing all aspects of a project from inception to operation. The more complex a project, the more essential and impactful value engineering becomes. Its primary objectives include identifying and eliminating unnecessary costs, enhancing timeline efficiency, improving quality, and modifying or eliminating other factors that might diminish value—all without compromising the project's core functions. A deep understanding of the subject, along with out-of-the-box thinking and creativity, serve as fundamental pillars of effective value engineering.

The top-down construction method, with its inherently complex, integrated, and multidimensional nature, offers significant opportunities for creative approaches to enhance efficiency and generate value. As the method continues to evolve, further refinements and optimizations will undoubtedly emerge, leading to even greater advancements. The development of the SPW approach, widely used in top-down construction in Iran, is itself a result of innovative thinking and value-driven management tailored to geotechnical conditions.

This is an indication that many untapped opportunities remain that can range from large-scale improvements in design and construction methodologies to project-specific optimizations. Potential areas for innovation include refining calculation methods and applied force assessments, enhancing soil-structure interaction analysis, optimizing structural systems, improving slab construction techniques, and developing more efficient excavation and soil disposal methods.

5.10 The Top-down Method in Conjunction with Other Excavation Methods

Depending on the conditions of adjacent structures, economic considerations, and technical requirements, the use of combined excavation methods may sometimes be necessary. Such integrations are not limited to the top-down construction method; hybrid approaches are common practice in many excavation projects regardless of the adopted method. For instance, in a given project, a truss retaining system may be employed on certain sides of the excavation, while methods involving controlled encroachment into neighboring properties might be used on others. This section presents an example of such combined methods, though the possibilities are not limited to these cases. The most suitable approach should always be determined based on the specific geometry and conditions of each project.

As mentioned earlier, many excavation methods that encroach on neighboring properties have led to legal challenges in Iran in recent years as they might entail boundary violations. Consequently, there is a growing preference for stabilization techniques that remain entirely within the project site. However, this restriction typically does not apply to projects adjacent to alleys or streets, where encroaching methods are often preferred by developers due to the advantages they offer. In such cases, to simplify excavation and avoid the complexities of full top-down construction, soil excavation with a stable slope can be carried out on the sides where encroachment is feasible. This approach allows for soil nailing, enabling the structure to be built from the bottom up while integrating with the top-down method to complete the excavation efficiently. Figure 5.13 provides an example of this approach.

In some cases, when encroachment methods are not permitted, the distance between the project site and the neighboring building may limit the feasibility of using soil nailing or anchoring systems with long lengths, as these elements would extend into adjacent properties. In such situations, a stable excavation depth can be determined based on the allowable soil nailing depth, allowing the upper portion of the excavation to be stabilized using soil nailing. The lower portion can then be executed using the top-down method. This approach enables a significant portion of the excavation to be completed without the full complexities of top-down construction while allowing substantial excavation progress. Additionally, it provides the opportunity to prefabricate top-down columns, further streamlining the process. A schematic illustration of this approach is presented in Figure 5.14.

These combined methods can vary widely based on the excavation geometry, and this section presents only a few examples (Figures 5.13 to 5.15).

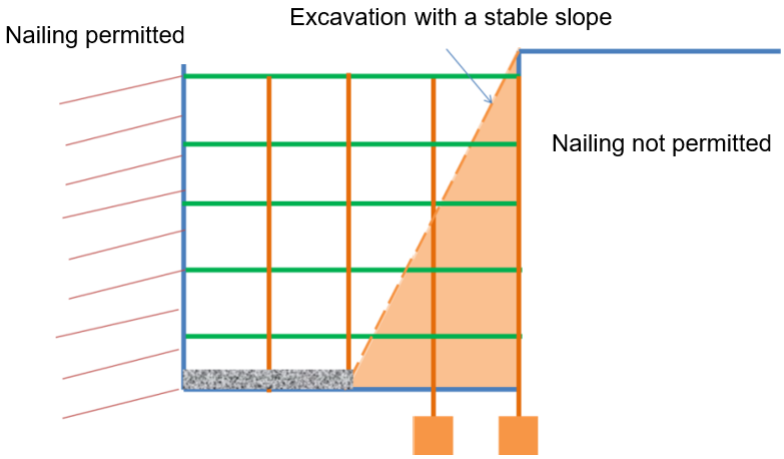


Figure 5.13 Combination of nailing and top-down methods (Example 1)

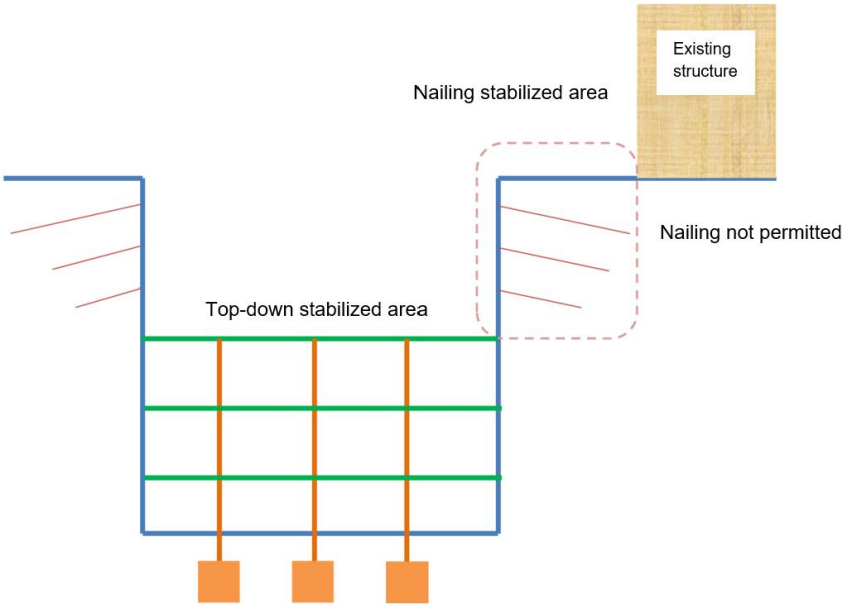


Figure 5.14 Combination of Nailing and top-down methods (Example 2)



Figure 5.15 Combination of anchoring and top-down methods at a construction site in Tehran, Niavaran

As previously mentioned, reducing the volume of excavation under the slab in top-down construction projects can have a significant impact on both cost and project timelines. One effective strategy to achieve this is integrating the island excavation method with the top-down method. A schematic representation of this method is shown in Figure 5.16. In this case, the combined approach involves the following steps:

1. Excavation with a stable slope down to the foundation level, followed by the construction of the central core up to ground level using the bottom-up method.
2. Sequential excavation of the side platforms (berms), along with the construction of the slab and side walls, using the top-down method.
3. Completion of the foundation for side sections.
4. Construction of the remaining columns using the bottom-up method.

In this approach, the columns embedded in the platforms can also be constructed using the top-down method. As noted earlier in this section, these examples are intended for illustration purposes only. In practice, combined methods are not confined to the examples provided and should be tailored to the specific geometry and soil conditions of each project.

The top-down and bracing methods share many similarities, making their integration a common practice in numerous projects. In some cases, bracing is employed to enhance stability in large openings within the slab diaphragm.

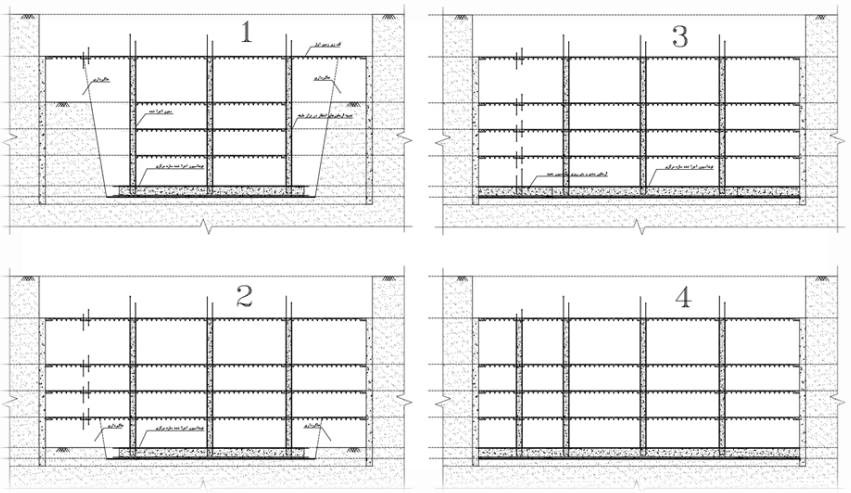


Figure 5.16 Combination of island and top-down methods

5.11 Common Misconceptions

This section examines common misconceptions in top-down construction that are frequently observed in project workflows. These errors will be categorized into three main areas: managerial, execution-related, and design-related. Each category will be introduced with a common misconception, followed by a detailed analysis of specific issues.

5.11.1 Managerial Misconceptions

From a managerial and economic perspective, the following common mistakes are often observed in the selection and implementation of the top-down construction method:

- **Top-down construction is more time-consuming compared to other methods.** This is not necessarily true. With effective management of excavation operations, the selection of appropriate machinery for soil removal, and the simultaneous construction of above-ground and below-ground floors, the overall project timeline may not be

significantly longer than other methods. In some cases, it can even be shorter.

- **The top-down construction method is more expensive than other methods.** This is not true. In fact, it typically requires less material for both excavation and structural work compared to conventional methods. Given the significant increase in material costs in recent years, top-down construction remains a competitive option—especially when compared to methods that rely heavily on temporary structures for excavation support.
- **The top-down construction method is only applicable to steel frame structures.** This statement is incorrect, as there are numerous examples of top-down construction successfully implemented in concrete structures, with specific execution considerations.
- **The top-down construction method leads to lower quality of execution in structural components such as columns and structural walls.** This statement is inaccurate. Any structural element that meets quality control requirements will maintain its intended quality. In top-down construction, it is essential to implement a robust quality control system to ensure compliance with standards and maintain construction quality.

5.11.2 Execution-related Misconceptions

Misunderstandings related to the execution of the top-down construction method can lead to unnecessary complications, added costs, and design limitations.

- **The top-down construction method requires special execution instruments.** This statement is not accurate, as this method does not necessitate unique tools or machinery. The instruments commonly available in general

workshops for conventional steel and concrete structures is sufficient for its implementation.

- **Midspan (auxiliary) columns must be made from steel.** This statement is incorrect. There are many examples of both midspan (auxiliary) and main columns being constructed using concrete. The choice of material is determined by the designer based on project requirements, and there is no strict requirement to use a specific material for these columns.
- **Floors in the top-down construction method must necessarily be of the beam-slab type.** This statement is incorrect. In reality, any type of floor system can be implemented in this method. Each system comes with its own technical, economic, and time-related advantages and limitations. The appropriate floor system for each project is determined based on the judgment of the designer and contractor, considering project-specific requirements.
- **In the top-down construction method, a lower gallery must be excavated in the shaft where the columns are installed.** This statement is not accurate. A gallery is excavated only when visual inspection is necessary for the installation of columns using connectors. In many cases, there is no need to construct a lower gallery.
- **In the top-down construction method, the column must be installed on the pile using a base plate.** This statement is not accurate. In some cases, the column is installed as a floating column with a shear key anchored into the pile, and in many instances, including most global practices, this method is preferred.
- **In the top-down construction method, the excavation of walls must necessarily be done all at once.** This statement is not accurate and is entirely dependent on the execution

method proposed by the designer. Depending on the work fronts and soil conditions, excavation can be carried out using the staggered or continuous approach.

- **In the top-down construction method, soil removal from the project site must be carried out using a tower crane.** This statement is not accurate. Soil extraction can be performed using a conventional or long-arm excavator, a lattice boom crane equipped with a clamshell bucket or suitable container, a telescopic boom crane with a clamshell bucket, or other instruments introduced in Chapter 3 of this guide, depending on the access conditions and geometric constraints of the project.

5.11.3 Design-related Misconceptions

Top-down construction design is a specialized skill. Some of the misconceptions common in top-down construction design are the following:

- **Only designers with a specialized skillset can carry out top-down design.** This statement is not accurate. In the top-down construction method, the designer must have expertise in construction techniques, geotechnical engineering, and structural design. This responsibility may be handled by a single individual or a team of engineers working collaboratively. An engineer qualified to design top-down structures should ideally hold a master's degree in geotechnical engineering, possess a strong foundation in structural engineering at the undergraduate level, have experience in structural design, and be well-versed in the top-down construction method. A deep understanding of soil-structure interaction is particularly crucial in ensuring the success of top-down construction projects. Many of

these skills are necessary for a successful design undertaking, regardless of the adopted method.

- **The designer of the top-down excavation method must be the same as the structural designer.** This statement is not accurate. The optimal approach is to first design the structure independently of the excavation method, ensuring that all components are clearly defined. The top-down construction designer should then develop the excavation plan while verifying that the structural components can withstand the demands of the execution method. If any elements are found to be inadequate, appropriate reinforcements should be introduced to ensure structural integrity throughout the construction process.
- **In the top-down construction method, the empty space between the shaft and the column must always be filled with concrete.** This is not necessarily correct. The designer may specify different materials, such as a soil-cement mixture with an appropriate cement content, which can be both practical and effective in meeting project requirements.
- **In the top-down construction method, the design should rely on mid-span columns being supported by the soil wall.** Although this approach is recommended, it is not mandatory. The designer may have planned the excavation sequence so that columns are not required to withstand lateral soil pressure. Instead, these columns may serve only as supplementary elements, assisting the main structural columns in redistributing applied forces during construction. In such cases, supporting the columns against the excavation wall is not necessary.
- **In the top-down construction method, it is necessary to perform seismic analysis.** This statement holds true for

structures with a long construction period (exceeding two years). In most top-down construction projects, excavation is a short-term phase, and assessing structural components for seismic forces during this stage is generally unnecessary. However, the main structure, regardless of the excavation method, must undergo a seismic assessment using appropriate and code-compliant methods.

- **Geotechnical modeling in the top-down construction method must be three-dimensional.** This is necessary under complex geometric conditions and when complex networks of openings and other structural components are present. In other cases, two-dimensional geotechnical modeling is adequate.
- **The design and assessment of structural components must necessarily be done using structural design software.** This statement is not always correct. What is essential is understanding the shear forces, axial forces, and bending moments generated in the structural components in the context of the adopted construction method. If the construction conditions and the type of components allow for the verification of the generated forces through geotechnical modeling, the necessary checks can be performed manually. However, in complex geometric conditions, where there is a risk of buckling in certain components (which geotechnical software cannot simulate) and under the assumptions of two-dimensional models, it is recommended that structural assessments be conducted using structural design software, incorporating soil-structure interactions, as determined by the designer.
- **Creating multiple files of different construction stage models in ETABS for the design and construction using the top-down method is sufficient.** This approach is not

correct and can lead to erroneous results. In top-down construction design, one of the most comprehensive modeling approaches is staged construction nonlinear analysis. This method enables the simulation of soil-structure interaction behavior and construction phases within a single computational file, providing a more realistic representation of the staged construction process. When using conventional modeling with multiple ETABS models, careful attention must be given to critical modeling details. The boundary conditions at the base of the retaining walls should accurately reflect in-situ construction conditions rather than being assumed as fixed. Additionally, at each stage, the boundary conditions at the base of the columns should be defined to ensure proper restraint at an appropriate depth below the retaining wall for the corresponding excavation phase. It is important to note that no single numerical software currently provides a fully integrated simulation of both soil and structural behavior. Relying solely on either structural or geotechnical software may lead to the omission of critical data. Therefore, for a more accurate and reliable design, it is recommended to use the combined results of sequential construction modeling from both structural and geotechnical software.

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Appendix I

Excavation Methods

Appendix 1: Excavation Methods

A1.1 Introduction

This appendix explores common alternatives to the top-down method. By comparing the top-down method with other excavation techniques, readers will gain valuable insights and can make more informed decisions for their projects.

Retaining systems for excavation walls offer different capabilities and features. In highly loose soils, these systems must be implemented before excavation begins. Suitable options for such conditions include concrete retaining piles and shafts near the excavation, diaphragm walls, and steel sheet piles—provided their use is feasible and the risks associated with driving are carefully managed.

In soils with relatively good stability, retaining systems can be constructed progressively alongside excavation. However, careful monitoring of excavation-induced deformations and their impact on adjacent structures' foundations is essential. If deformations exceed allowable limits, retaining systems (support structures) should be established before excavation.

When assessing excavation stability, the selection and design of retaining systems should account for various factors, including soil type and stratification, shear strength and deformation parameters, excavation depth and width, groundwater and surface water conditions, nearby structures and their construction methods, surcharges such as street traffic, and whether the excavation is for short-term or long-term use. Additionally, potential failures and deformations during excavation—such as soil sliding, settlement and displacement of adjacent buildings, collapse, floor heaving, sand boiling (in cases of high groundwater levels), vibration-related impacts on nearby structures, and seismic

effects for non-temporary excavations—must be carefully evaluated.

The following excavation and stabilization methods will be introduced and discussed:

- Stable slopes or stepped slopes
- Truss support structures
- Strut support structures (compressive elements)
- Anchored sheet pile systems
- Pile and lagging systems
- Concrete retaining walls
- Soil nailing for reinforcement
- Soil anchoring systems
- Island excavation method

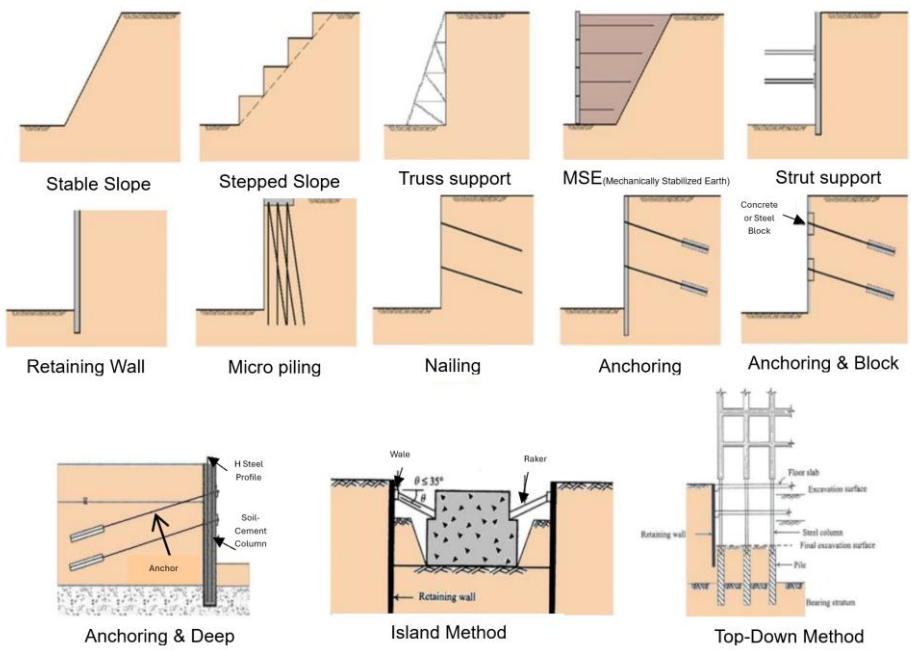


Figure A1.1 Excavation and stabilization methods

A1.2 Stabilization Using Stable Slopes or Stepped Slopes

A safe method for stabilizing excavation walls is using stable or stepped slopes. When sufficient space is available around the excavation, a sloped design can be implemented. This approach eliminates the need for specialized equipment, but it increases lateral soil deformations. Moreover, due to the substantial space required, it is generally not a viable option in urban environments.



Figure A1.2 Stable slopes or stepped slopes stabilization method

Table A1.1 Stable slopes or stepped slopes stabilization method characteristics

	Advantages	Disadvantages
Characteristic	Cost-effectiveness, simplicity	Requires an external execution area beyond the excavation
Comparison with the top-down method	Greater simplicity	Encroachment on adjacent properties

A1.3 Stabilization Using Truss Support Structures

This method is commonly used for support structures in urban areas and shallow excavations due to its simplicity, non-reliance on specialized equipment or high expertise, and relative flexibility. It is recommended for excavations with up to two basement levels and a width-to-depth ratio greater than 1.5.

The advantages of the truss support structure include the possibility of recycling some of the materials, no encroachment on adjacent properties, adaptability to drainage systems, and suitability for projects with irregular geometries.

Its disadvantages include taking up significant space (which complicates execution), limited applicability for deep excavations, the need for partial manual soil removal, difficulties in excavating and constructing the main structure, challenges in execution, reduced waterproofing quality, inability to increase excavation depth during or after excavation, issues with trusses in concave corners, and an apparent simplicity that may lead to involvement from unqualified individuals.

The use of worn-out components and castellated sections in constructing truss support structures is not recommended, although the use of used sections in good condition may be acceptable.

A1.3.1 Truss Support Execution Steps

Figure A1.3 provides a schematic representation of truss support execution steps.

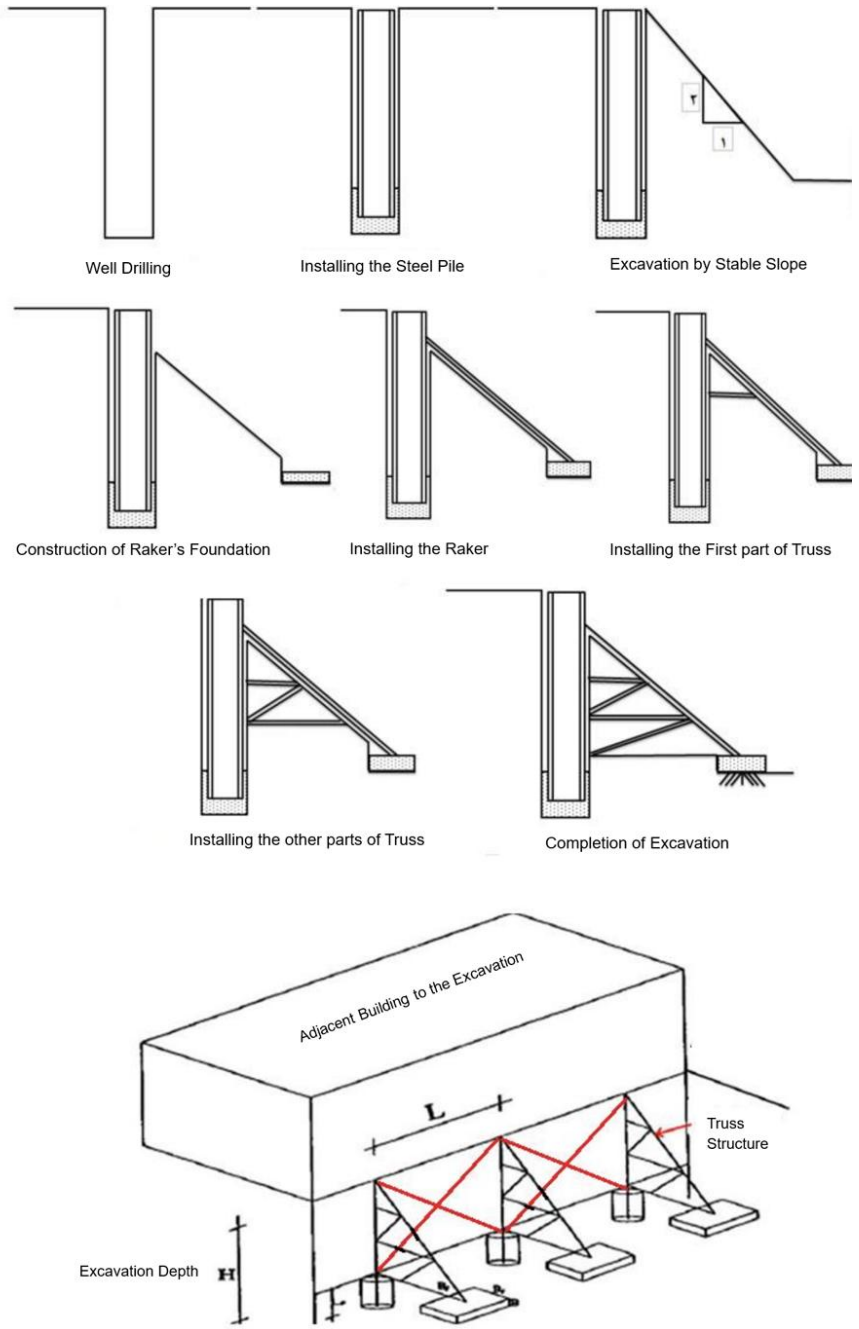


Figure A1.3 Truss support execution steps

This method involves replacing the excavated soil with a steel structure while stabilizing the surrounding soil. First, vertical shafts are drilled at specific points around the site. Next, reinforcement cages for the piles are placed in their designated locations. The vertical members are then positioned inside the shafts, and concrete is poured at the base. The shafts are then filled with lean concrete. Afterward, the soil is excavated with a stable slope across the entire excavation area. Finally, the inclined members are installed to complete the support structure.



Figure A1.4 A truss support structure

Table A1.2 Truss support stabilization method characteristics

	Advantages	Disadvantages
Characteristic	Common in small excavations High flexibility Recyclability of truss materials Simplicity No need for specialized expertise or specific equipment	Relatively slow execution The truss can be cumbersome. Potential need for manual excavation Low safety
Comparison with the top-down method	Greater simplicity	Lower safety

A1.4 Stabilization Using Strut Support Structures (Compressive Elements)

In urban environments, where the use of truss retaining structures is restricted due to dimensional constraints or excessive depth, or where tieback stabilization is not feasible due to obstructions, the strut method offers a suitable alternative for excavation stabilization. This method relies on a combination of horizontal (struts) and diagonal (braces) metal components, along with vertical elements such as steel piles, concrete walls, shotcrete, concrete piles (separate or continuous), steel sheet piles, and diaphragm walls, to ensure stability and control deformations caused by excavation. Compression braces, made of metal or concrete, may also feature hydraulic functionality with adjustable lengths. The choice of vertical components depends on project conditions, design requirements, groundwater levels, soil type, and other relevant factors. Additionally, horizontal components with variable lengths can be designed in accordance with standard references, provided all executional requirements are carefully considered.

In the strut method, the soil pressure exerted on the excavation walls is transferred to the opposite wall through horizontal components. This method is commonly used for narrow trenches, such as canals, as well as deep excavations in urban environments. When designing and arranging the components of the strut system, it is advisable to maximize the structural degree of indeterminacy to minimize the risk of progressive failure.

This method is most suitable for excavation widths of approximately 10 meters up to a maximum of 15 meters. The spacing of structural components should be carefully adjusted to ensure both the necessary strength and stiffness for system stability while also considering executional feasibility.

Among its advantages for narrow excavations are its ability to limit deformations, compatibility with drainage methods, design flexibility, the possibility of reinforcement during and after

execution, independence from soil type, lower cost, faster construction speed, minimal space occupation, and non-reliance on specialized equipment.

However, this method also has several drawbacks, including challenges posed by uneven soil conditions on both sides of the excavation, the impracticality of using heavy machinery for deep excavations, inefficiency in large or deep excavation projects, considerable space occupation within the excavation area, difficulties in soil removal, challenges of main structure construction, relatively slow execution speed, limitations in environments with restricted access, challenges in waterproofing execution and quality control, restrictions in projects with irregular geometries, inability to extend excavation depth after initial completion and the requirement for skilled personnel and strict quality control, particularly for brace connections.

In long excavations or tunnels where struts are used for lateral support, it is advisable to utilize a continuous horizontal integrating beam (waler beam) rather than directly connecting the horizontal bracing to the retaining structure. The connection between the horizontal bracing and the continuous beam should be designed to prevent movement along all three principal axes.

To maintain structural integrity, components should not be placed at the roof level or integrated with the main structure's roof. In some cases, the concrete floor slab may serve as a strut, provided the design accounts for adequate stiffness and resistance to both lateral loads from the retaining structure and vertical loads from construction machinery.

The continuous beam must possess sufficient strength and stiffness to effectively transfer forces from the retaining structure to the strut. Its vertical spacing should be adjusted to provide the required safety of the retaining system—ensuring adequate strength and stiffness—while accounting for construction feasibility. The beam's dimensions should be selected to

accommodate the connection details of the strut, with a larger cross-section at contact points for enhanced load distribution.

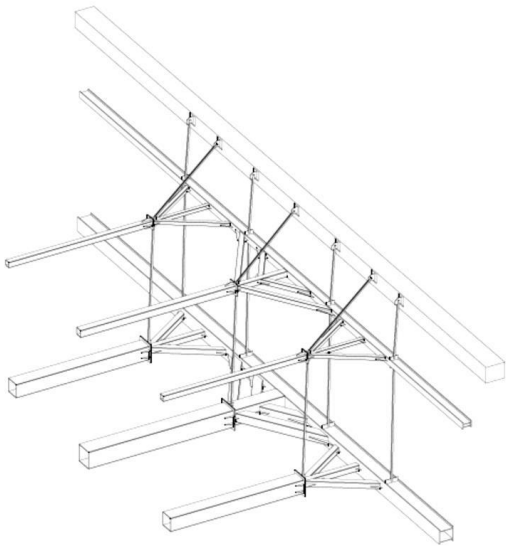


Figure A1.5 The connection between the horizontal bracing and the continuous beam to the pile head



Figure A1.6 Excavation stabilization using struts

Table A1.3 Strut stabilization method characteristics

	Advantages	Disadvantages
Characteristic	High execution speed Suitable for narrow excavations No encroachment on adjacent properties Recyclability of strut materials	Space challenges High costs The need for precision in connection design and execution High expertise requirements
Comparison with the top-down method	Higher speed	Lower safety

A1.5 Stabilization Using Anchored Sheet Pile Systems

In this method, steel sections are driven into the soil along the excavation walls using pneumatic hammers or vibration. They are then connected through various tongue-and-groove joints, forming a continuous retaining wall. This technique minimizes space occupation within the excavation site, which makes it suitable for constrained sites. Sheet piles are generally used as temporary structural elements. Their primary application is in narrow excavations for pipelines, tunnels, subways, and similar underground structures.

The implementation begins with driving sheet piles into the sides of the excavation followed by soil removal. This cycle continues until excavation reaches the designated depth. Next, waler beams are installed at the waist level of the sheet piles. Vertical compression braces (struts), positioned perpendicular to the sheet pile plane, are then connected to these horizontal struts—either from the front or by anchoring them into the soil at the back. Excavation continues in stages, with additional struts or anchors installed at each level as needed.

In narrow excavations with stable soil conditions, sheets, waler beams, and struts can be made of wood. However, in wider excavations with unstable soil conditions, using metal materials is inevitable.

Sheet piles can be flexible. The soil in front of the sheet pile can be in a passive state, resisting pressure, while the soil behind it applies active pressure. If the sheet pile is embedded sufficiently deep into the soil, it is considered fixed at the base, generating passive resistance. However, if the embedment depth is insufficient, the base behaves as a hinge, and the sheet pile is referred to as a free-base sheet pile.

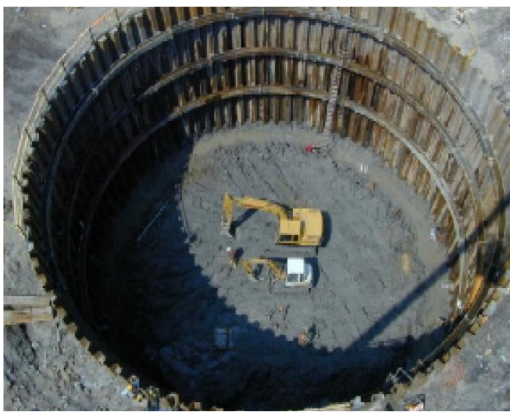


Figure A1.7 Excavation stabilization using sheet piles

A1.6 Stabilization Using Concrete Retaining Walls

A retaining wall is a concrete structure designed to support and stabilize soil, preventing its collapse. There are several types of concrete retaining walls:

- Gravity walls
- Cantilever walls
- Counterfort walls
- Crib walls
- Semi-gravity walls
- Diaphragm walls

Table A1.4 Sheet pile stabilization method characteristics

	Advantages	Disadvantages
Characteristic	High safety when anchored to the back Suitable for long excavations Appropriate for conditions with high groundwater levels	Special equipment requirements Suitable only for narrow excavations High expertise requirements
Comparison with the top-down method	Better space management	Higher costs Not applicable in gravelly or hard soils

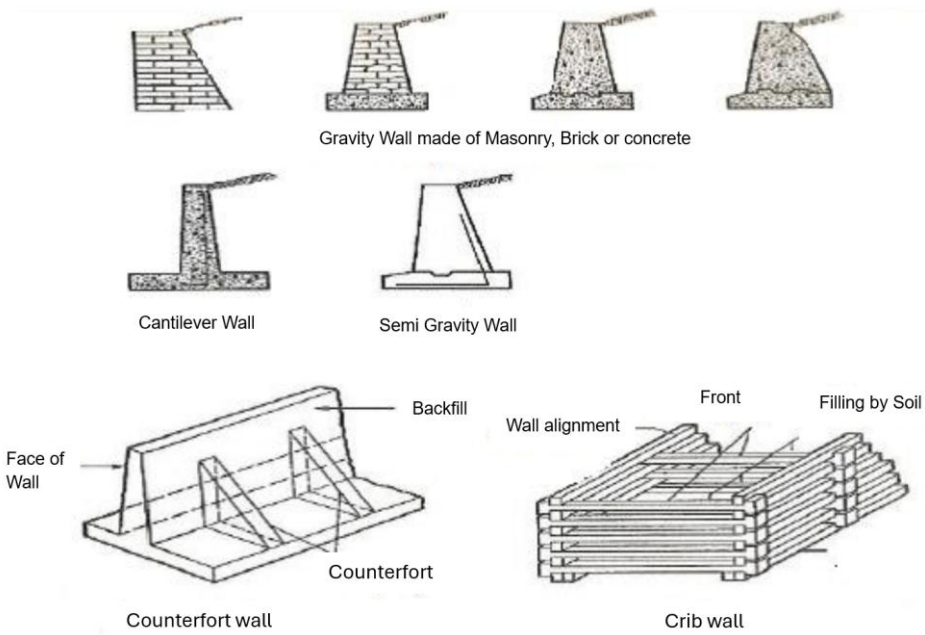


Figure A1.8 Retaining wall types made of reinforced concrete or masonry materials

Many of these concrete walls are not suitable for excavation support, as they cannot be constructed before soil removal to prevent collapse. However, cantilever and anchored walls can be implemented using various construction techniques, including diaphragm walls, piles arranged in different configurations, or even non-concrete alternatives such as soil-cement walls created through high-pressure injection (jet grouting) or deep mixing methods.

A1.6.1 Diaphragm Walls

Slurry walls, also known as diaphragm walls, are a type of reinforced concrete retaining wall constructed by excavating deep trenches. In this method, specialized drilling equipment is used to excavate the perimeter of the site while simultaneously filling the trench with bentonite slurry to prevent collapse. Once excavation is complete, a reinforcement cage is placed inside the trench, which is then filled with concrete. This method is commonly used in deep excavations and tunnels. Its application and execution process in the conventional top-down construction method is detailed in Chapter 3 of this guide.

Beyond top-down construction, slurry walls serve as a reliable and robust stabilization system when combined with cross-braced struts or tie-back anchors. This method is particularly beneficial in high groundwater conditions, offering significant structural and executional advantages.



Figure A1.9 Excavation stabilization using diaphragm walls

Table A1.5 Diaphragm wall stabilization method characteristics

	Advantages	Disadvantages
Characteristic	High safety when anchored to the back	High cost for small projects Inability to execute in confined urban spaces Special equipment requirements High expertise requirements
Comparison with the top-down method		Specialized equipment and technology requirements Potentially higher costs

A1.6.2 Pile Walls

Another method for constructing retaining walls involves installing a series of closely spaced piles. These piles, known as soldier piles, can be arranged with gaps between them, placed tangent to each other (tangent piles), or interlocked with overlapping sections (secant piles) to enhance structural integrity. When soil conditions are highly favorable, gaps between the piles may be acceptable. Otherwise, tangent or secant pile arrangements are used to improve stability. Additionally, these piles can be

supported with tie-back anchors or cross-bracing, similar to diaphragm walls, to enhance their load-bearing capacity and overall effectiveness (Figure A1.10).

These piles are installed at predetermined intervals around the perimeter of the excavation site and can be made from various structural materials, including steel, concrete, or timber. Concrete piles may be either precast or cast-in-place. In some cases, the piles act as cantilever beams to resist lateral soil pressure, requiring an embedment depth of approximately 30% of the total pile length for stability. Once the piles are in place, excavation can proceed. However, if additional support is needed, bracing along the excavation or cross-bracing should be implemented to enhance stability.

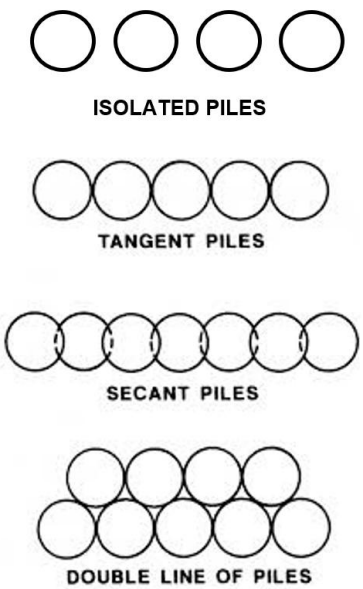


Figure A1.10 Excavation stabilization using different configurations of pile walls



Figure A1.11 Excavation stabilization using tangent piles

If the piles are spaced apart, the gaps can be filled with materials such as timber, precast concrete segments, or shotcrete to prevent surface soil collapse. Additionally, piles can be constructed from various materials and cross-sections, including steel H-sections, depending on the project requirements and soil conditions.

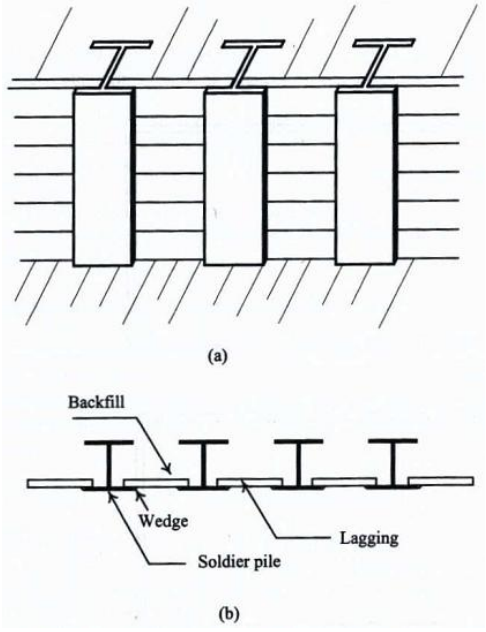


Figure A1.12 Excavation stabilization using spaced soldier piles with pile lagging

In this method, steel beam sections are driven into the ground to depths exceeding the planned excavation. The spaces between the beams are then filled with timber planks, typically 5 to 10 cm thick, positioned behind the front flange. Bracing may be added at specific levels if needed.



Figure A1.13 Excavation stabilization using spaced soldier piles

Table A1.6 pile stabilization method characteristics

	Advantages	Disadvantages
Characteristic	Offering most of the benefits of diaphragm walls without requiring complex equipment	Time-consuming At great excavation depths, requires tie-back anchors, encroaching on adjacent lots
Comparison with the top-down method		Needs more space Work plane irregularities

A1.6.3 Soil-cement Pile Walls

Soil-cement pile walls function similarly to concrete piles, but instead of concrete piles, they utilize soil-cement piles. The advantage of soil-cement piles lies in their simple and rapid construction, achieved through deep soil mixing or jet grouting techniques.

In this method, a uniform column is created in situ by mixing soil with stabilizing materials such as cement and lime. Overlapping these columns forms a sturdy wall for excavation stabilization. Although the stiffness of a soil-cement wall is significantly lower than that of concrete—making it less suitable for deep excavations—the speed and ease of construction make it a highly effective solution in many cases.



Figure A1.14 Excavation stabilization using soil-cement columns



Figure A1.15 Excavation stabilization using anchored soil-cement columns

In the deep mixing method, the mixing equipment penetrates the ground using its arms while simultaneously applying pressure and rotation. Once it reaches the desired depth, the downward thrust stops, and the mixer begins to move upward in stages. During this upward movement, a slurry is injected at high pressure through nozzles at the tips of the arms. After each one-meter increment, the mixer pauses and rotates for approximately one minute at the same level to ensure thorough mixing of the slurry with the soil. The exact penetration depth, injection pressure, and mixing duration may vary depending on the equipment type and soil conditions.

Another method for constructing soil-cement columns is jet grouting. In this technique, a drilling bit equipped with a specialized injection nozzle rotates while applying pressure to drill a hole approximately 10 centimeters in diameter, reaching the desired depth. At this point, high-pressure cement slurry injection begins as the bit slowly rotates and is gradually withdrawn. This process erodes and mixes the surrounding soil within a specific diameter, forming a soil-cement column. To enhance bending resistance, H-shaped sections can be driven into the freshly formed

columns before the soil-cement mixture sets, reinforcing the structure.

Table A1.7 Soil-cement pile method characteristics

	Advantages	Disadvantages
Characteristic	High construction speed Non-intrusivity Cost reduction for large volumes Suitable for conditions with high groundwater levels	Typically, only economically viable for shallow excavations Drilling challenges in hard soils Quality control challenges
Comparison with the top-down method	Providing adequate workspace	Significant depth limitations

A1.7 Stabilization Using Wall Nailing

The nailing method also referred to as the tie-back method, is another technique for stabilizing deep excavation walls. This method reinforces the soil by effectively stitching soil masses together using tensile anchors. The process begins with excavation to a level slightly below the designated position of the reinforcement bar. Next, holes are drilled at an angle of 10 to 25 degrees, into which reinforcement bars are inserted. These holes are then filled with cement grout to secure the bars in place. A steel mesh is applied to the wall, and shotcrete is applied. This sequence is repeated progressively until the excavation reaches its final depth.

The nailing method is best suited for excavations ranging from 8 to 15 meters deep in well-graded, relatively dense granular soils with an SPT value greater than 30, stiff fine-grained soils with an SPT value greater than 10, and weathered rocks without weak planes.

Sandy soils containing 15 to 20% fines are generally considered ideal for stabilizing excavation walls using this technique. If an excavated section, standing 1 to 2 meters high, can maintain

vertical or near-vertical stability for one to two days—even after becoming fully saturated—without collapsing in bulk, and if drilled holes remain stable for several hours (allowing sufficient time for reinforcement placement and grout injection), then the use of a protective casing is unnecessary. Under these conditions, the nailing method is not only technically feasible but also highly cost-effective.

The nailing method offers several advantages for stabilizing excavation walls, including relative flexibility, the ability to accommodate significant and acceptable settlement, and permissible deformation and bending of the nailed wall. Due to its flexibility, it performs well under seismic loads and is a cost-effective solution. Additionally, this method allows for adjustments and repositioning when encountering underground utilities and enables modifications to the wall design without compromising safety—an advantage over other methods like anchoring.

Other key benefits include high construction speed, applicability in hard-to-access environments, minimal occupation of the project site's space, and no interference with the main structural elements. This method also supports permanent stabilization, integrates well with drainage and waterproofing systems, and allows for reinforcement during and after construction. Moreover, it is suitable for projects with irregular geometries, can be used in partially completed or unstabilized projects, and offers the flexibility to increase excavation depth during or after excavation.

The nailing method has several disadvantages, including encroachment on neighboring properties, limited control over displacements, and construction challenges in areas with underground facilities. It also requires specialized expertise for nail installation and may lead to water leakage issues in regions with a high groundwater table.

Other drawbacks include the potential for significant displacements, particularly near sensitive structures, as well as the need for specialized equipment and machinery. The implementation of the method can be challenging in environments with underground voids and necessitate experienced contractors. Considerable noise pollution can also cause problems. Additionally, there is a risk of damage to nearby green spaces and wells.

A key concern is that, due to the passive nature of the components in this method, resistance forces develop only after soil movement occurs, which can result in undesirable deformations. Furthermore, drainage systems must be carefully designed and implemented to ensure the long-term stability of the excavation.



Figure A1.16 Excavation stabilization using wall nailing

Table A1.8 Wall nailing method characteristics

	Advantages	Disadvantages
Characteristic	Suitable working space Cost-effectiveness (feasibility in large projects) Frame construction speed	Encroachment on neighboring properties
Comparison with the top-down method	Availability Speed Better space management	Encroachment on neighboring properties

A1.8 Stabilization Using Anchored Walls

One of the most widely used excavation stabilization techniques is the tie-back anchoring method. This ground anchoring approach involves drilling multiple boreholes into the surrounding soil, inserting reinforcing elements (such as strands or rebars), and injecting grout to secure them. This process enhances the shear resistance of the soil along the potential failure surface, improving excavation stability effectively.

The primary advantage of the anchoring method over nailing is the application of a prestressing force to the reinforcing elements, which helps minimize deformations caused by excavation. In contrast, nailing relies on deformation in the soil mass to activate resistance forces, meaning some movement must occur before the system becomes fully effective. Additionally, prestressing ensures optimal performance of the anchoring components from the outset, enhancing overall stability and control.

Other advantages of the anchoring method include reduced displacements, ensuring the reliable performance of all anchors, and the ability to stabilize deep excavations. This method requires minimal space, does not interfere with main structural components, and allows for fast execution. It also offers the possibility of permanent stabilization, compatibility with drainage and waterproofing systems, and design flexibility, enabling

reinforcement during and after construction. Furthermore, it is suitable for irregularly shaped projects, can be applied to partially completed or unstabilized excavations, and allows for excavation depth adjustments even after the process is completed.

Some drawbacks of the tie-back anchoring method include encroachment on adjacent properties, the need for longer drilling lengths compared to soil nailing, and the requirement for specialized equipment and machinery. Other challenges include limitations due to underground voids, the necessity of experienced and specialized contractors, noise pollution, and the risk of damage to nearby green spaces and wells.

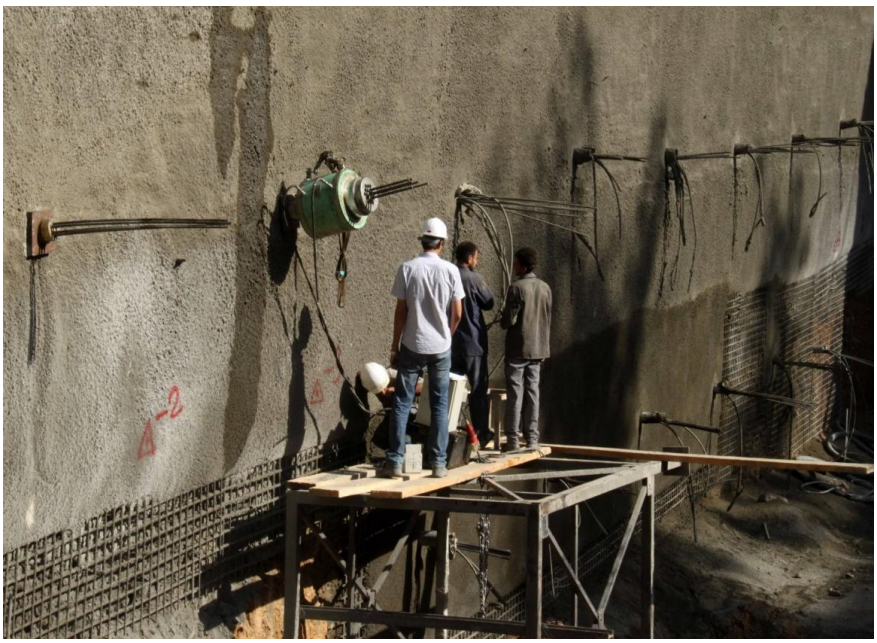


Figure A1.17 Excavation stabilization using wall anchoring

In the tie-back anchoring method, appropriate components should be used to control the anchor force at the anchor head and ensure its effective transfer to the soil. These components may include vertical elements, concrete or steel pads, or a combination of both. Vertical elements are typically made of steel sections or steel piles, while concrete pads are reinforced concrete blocks that can be

either precast or cast-in-place. Steel pads consist of engineered metal plates positioned beneath the anchor head. The behavior of these pads resembles that of isolated foundations, functioning as two-way slabs. The concrete pad must be designed to withstand punching shear and bending forces to ensure stability and proper load distribution.

This method is executed step-by-step, from the top to the bottom of the excavation. The stages of implementation and the working procedure are as follows:

1. The excavation process begins in stages (each stage typically 2 to 3 meters deep), reaching approximately 0.6 meters below the design level of a soil anchor.
2. Horizontal or inclined boreholes are drilled into the excavation wall using specialized drilling equipment.
3. Monobar or strand prestressing cables are placed inside the boreholes along with a polyethylene injection pipe. Grout is injected at the end to anchor the cable.
4. The cables are tensioned using a jack (applying prestress) and anchored at the surface.
5. Cement grout is injected into the boreholes.
6. After the concrete has set, the jacks are released.
7. Shotcreting is carried out to complete the process.

Table A1.9 provides a comparison of nailing and anchoring methods using vertical pile elements or pads.

Table A1.9 Comparison of nailing, block-anchoring, and pile-anchoring methods in deep excavations

	Method	Safety	Deformations	Costs	Execution time
1	Nailing	Minimum	Maximum	Maximum	Similar to 2
2	Pile-anchoring	Maximum	Minimum	Medium	Similar to 1
3	Block-anchoring	Medium	Medium	Minimum	Minimum

Table A1.10 Wall anchoring method characteristics

	Advantages	Disadvantages
Characteristic	Improvement of the soil's mechanical properties Suitable working space management Active method with control over deformations Fast execution of the structural framework	Encroachment on neighboring properties Requiring specialized labor and equipment
Comparison with the top-down method	Better space management	Encroachment on neighboring lots Use of materials exceeding what is necessary for the construction of the structure

A1.9 The Island Excavation Method

The island excavation method involves excavating the central part of the project site with sloped walls, constructing the central structure, and then removing the surrounding soil while using the central structure as support. This method typically begins with the construction of a retaining wall around the perimeter of the site, followed by the installation of horizontal struts that provide lateral

bracing against the retaining wall. Once these initial supports are in place, the central area is excavated with sloped sides. After reaching the desired depth, a portion of the structure is built in the center of the excavation, and a bracing system is established between the retaining wall and the newly constructed central structure. The sloped sections are then excavated in stages, allowing for the completion of the remaining structural components.
The execution steps in this method are the following:

1. Excavating the central part of the site with sloped walls.
2. Building the central portion of the structure in the middle of the excavation.
3. Installing anchor beams between the retaining wall and the central structure at an angle of less than 35 degrees.
4. Excavating the sloped areas and completing the construction of the remaining parts of the structure.

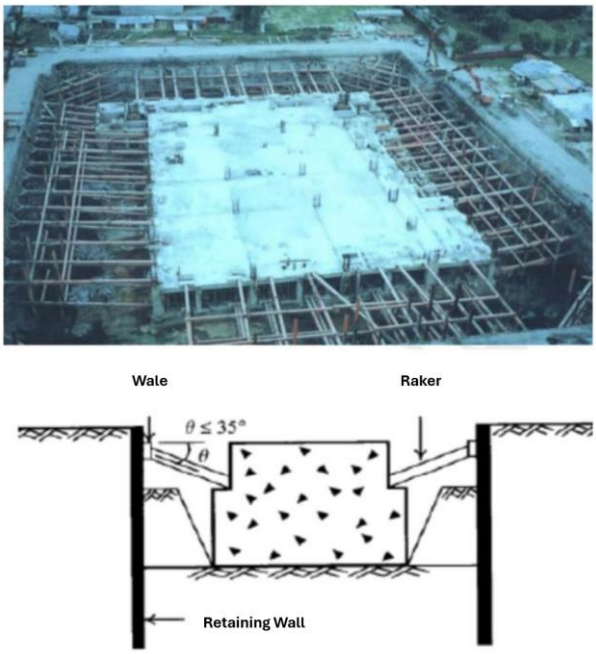


Figure A1.18 Excavation stabilization using the island method

Table A1.11 Table Island method characteristics

	Advantages	Disadvantages
Characteristic	Utilization of soil stabilization capacities Reduced costs Increased speed	Only suitable for large lots
Comparison with the top-down method	Reduced costs Increased speed	Not suitable for small lots

Appendix II

An Example of Structural Design

Appendix II: An Example of Structural Design

A2.1 Introduction

This appendix presents the design process of a reinforced concrete structure for a residential building project in District 1 of Tehran as an example of structural design for top-down construction. The 16-story concrete building of this project consists of five basement levels and eleven above-ground levels, situated on a plot of approximately 1,380 square meters. The basement levels have a building footprint of 60%, covering an area of 860 square meters. Excavation for the project reached a depth of 17.25 meters in one section and 18.5 meters in another.

Considering construction constraints and project design requirements, and following a technical and economic evaluation, the top-down construction method was selected for this concrete structure. This approach utilizes composite concrete columns (SRC) with a steel tube core filled with concrete (CFT). During construction, the concrete-filled steel core acted as the primary vertical load-bearing element, transferring column forces to the piles throughout the excavation and construction process. The adjacency conditions of this project are illustrated in Figure A2.1.

With the groundwater level situated at a depth of 19 meters from the project's reference level, the final execution steps were determined based on a comprehensive assessment of construction and geotechnical factors. These considerations included evaluating the load-bearing capacity of the piles in relation to the axial forces of the columns, analyzing force demands and structural design requirements, and refining the process through iterative geotechnical and structural design. After considering technical, economic, and construction constraints, these execution steps were established as the foundation for the final numerical modeling of both the structure and geotechnical conditions:

1. The wells for the main and auxiliary structural columns of the building are excavated to install the embedded columns using the top-down method at the project site.
2. The structural columns, along with the reinforcement cage for the individual piles beneath the columns, are fabricated and prepared.
3. The reinforcement cages for the concrete piles, with a diameter of 120 cm and lengths of 14.5, 5.5, and 8.6 meters, are installed according to the structural drawings inside the wells. The main structural columns are then suspended, plumbed, and installed at wellheads.
4. After plumbing and installing the columns with the wellhead components, the concrete for each pile is poured.
5. Once the concrete has set, the void between the well and the column is backfilled with suitable soil.
6. Following the installation of the structural columns as embedded elements in the basement levels, the second basement level is completed from the bottom up.
7. The first basement level is then completed from the bottom up.
8. While the construction of the third basement level and excavation are ongoing, the ground floor slab is concreted.
9. With the completion of the fourth basement level and the excavation, the structure is completed up to the first and second above-ground levels.
10. The fifth basement level is completed at the same time as the excavation operations.
11. Finally, the foundation is concreted.

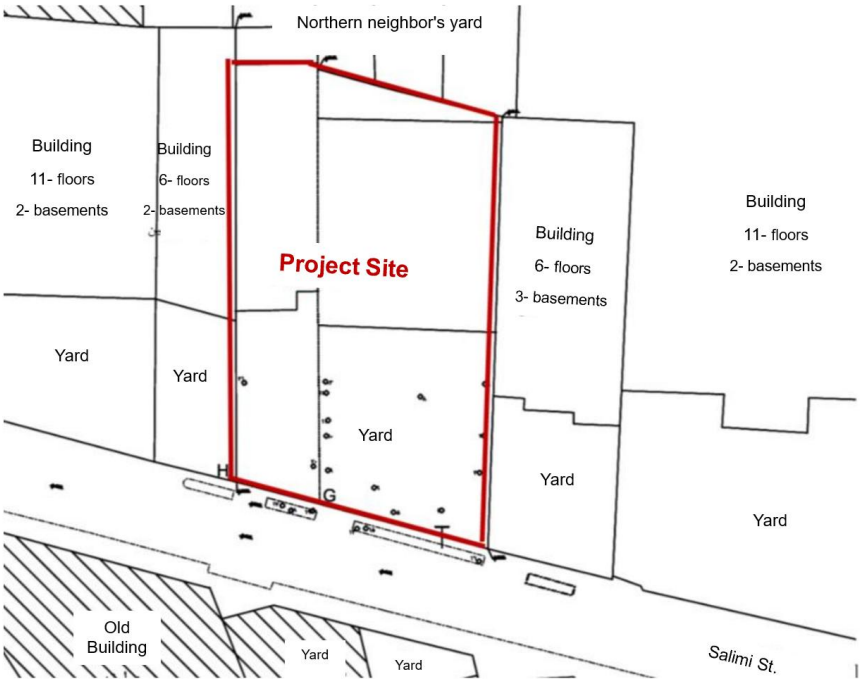


Figure A2.1 The site plan and adjacencies of the residential building project

Since the eastern and western adjacent structures have three and two basement levels, respectively, the initial excavation was carried out down to the second basement level of the western neighboring structure, reaching a depth of approximately five meters. Consequently, the drilling of wells and the installation of the central core commenced from this level, following the designated execution steps. This approach was also integrated into the structural and geotechnical construction phase modeling.

The design process presented in this appendix consists of two interdependent stages—structural and geotechnical—which have been developed in parallel through multiple iterations. As outlined in earlier sections of this guide, the design of the top-down construction method encompasses feasibility studies, construction method planning, geotechnical design, and structural design. However, this appendix does not cover the feasibility

study or the construction method design and only provides a brief overview of the geotechnical and structural design stages. The steps of this design process are as follows:

1. Geotechnical modeling, analysis, and design:
    a. Determining the load-bearing capacity of piles using empirical equations as well as two- and three-dimensional numerical modeling via the Plaxis software.
    b. Determining magnitude and distribution of soil pressure behind excavation walls under surcharge loads corresponding to each side using empirical equations and numerical modeling of construction stages.
    c. Determining equivalent soil spring stiffness or soil reaction coefficient, for use in modeling and staged structural design in the structural software (lateral and vertical stiffness of embedded columns and piles).
    d. Modeling structural frames in geotechnical software in two dimensions (multiple 2D frames along the East-West and North-South directions of the plan) or three dimensions, with 2D analyses being the focus of this project.
    e. Monitoring settlement and deformation of adjacent structures and ensuring overall stability of the excavation.
2. Structural modeling, analysis, and design:
    a. Modeling, analysis, and design of the structure for the conventional case without considering stage-by-stage construction to finalize the required volumes and sections for the structural design.

b. Modeling, analysis, and design of the structure for staged construction using the top-down method in the structural analysis software, ETABS.
c. Verification of pile settlement using equivalent soil spring data.

A2.2 Geotechnical Analyses

A2.2.1 Determining Isolated Pile Load-bearing Capacity and Equivalent Spring Using Numerical Method via Plaxis

Numerical modeling of a pile under axial compressive and tensile loads at the level below the foundation was conducted using the Plaxis 3D program, simulating soil removal after creating initial stresses and establishing the excavation floor level. The numerical analysis results are presented as a force-deformation graph in Figure A2.2 for a pile with a diameter of 120 centimeters and shaft lengths of 5.14, 5.6, and 6.84 meters, both with and without the bell-bottom derived from three-dimensional numerical analysis. The allowable compressive bearing capacity of a single pile was determined based on this graph, ensuring that the maximum settlement remained within allowable limits. Additionally, numerical graphs obtained from two-dimensional analyses in Plaxis 2D were extracted and evaluated. Alongside these numerical analyses, the allowable bearing capacity of the pile was ultimately assessed using empirical and classical soil mechanics relationships.

In this design approach, vertical force-displacement data is incorporated into the structural model by representing piles as equivalent vertical springs. To ensure safety, the vertical capacity generated between columns and the surrounding soil-cement is disregarded, assuming that the entire vertical axial load of the columns is transferred directly to piles. Finally, the forces acting on the piles in the staged construction model within the ETABS program, along with their corresponding settlements, are analyzed and compared against allowable values obtained from geotechnical assessments.

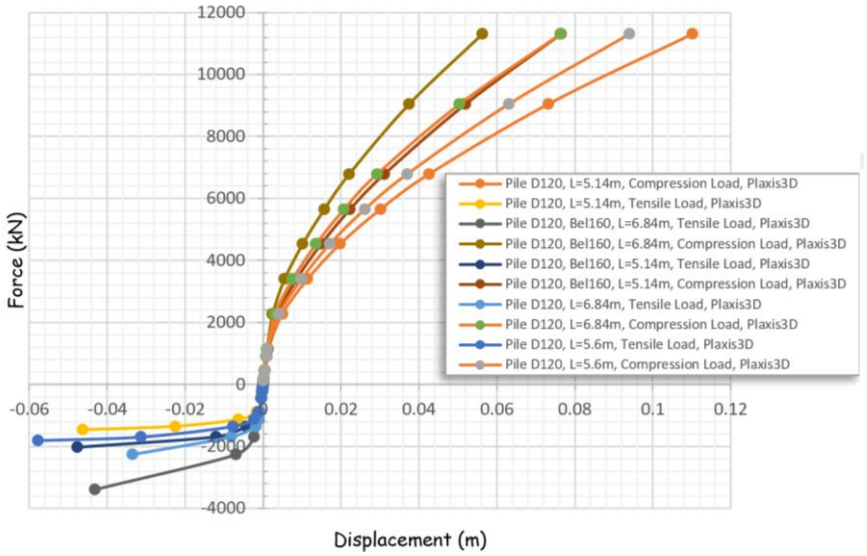


Figure A2.2 Force-deformation graph for a single pile with a diameter of 120 centimeters and shaft lengths of 5.14, 5.6, and 6.84 meters, with and without the 160-centimeter bell-bottom

A2.2.2 Soil Pressure Data for Structural Analysis

In the top-down construction method, the floor diaphragm is incrementally integrated with the columns, and the retaining wall is constructed from top to bottom based on geotechnical conditions. The overall system behavior resembles the soil pressure distribution of braced walls. Therefore, for structural analysis and preliminary estimation of materials, earth pressure was initially calculated using the empirical method proposed by Chebotaryov (1951) for sandy soils, as shown in Figure A2.3.

Next, refined geotechnical modeling and construction sequencing were performed in geotechnical software, incorporating the variables presented in Table A2.1. Through this numerical modeling, soil pressure diagrams were produced, accounting for two-dimensional soil-structure interactions and the conditions of adjacent buildings. The soil pressure distribution near the adjacent 8-story building on the western side is shown in Figure A2.3 as an example. Similar analyses were conducted for each side of the excavation walls, with soil pressure data extracted from

geotechnical numerical models and applied to the structural model in ETABS.

Table A2.1 Geotechnical characteristics of the site soil layers

Depth (m)	Unit Weight, γ (gr/cm³)	φ (°)	ψ (°)	C (kg/cm²)	E (kg/cm²)
0 – 0.6	1.8	30	0	0.10	300
0.6 – 15	1.85	32	2	0.05	500
15 – ∞	1.85	33	3	0.05	800

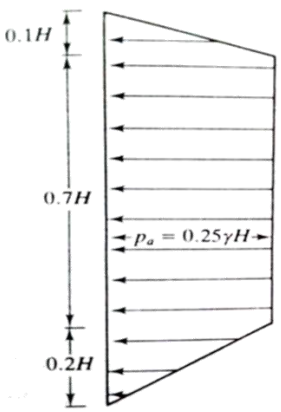


Figure A2.3 Soil pressure distribution in a braced wall (sandy soil), Chebotaryov (1951)

A2.2.3 Determining Soil Equivalent Horizontal Spring Stiffness under Lateral Load Using Plaxis 2D

The lateral stiffness of the pile (column embedded in soil) in this example was determined using a numerical approach. However, as mentioned before, these stiffness curves can also be derived from theoretical relationships. Using numerical modeling in Plaxis 2D, lateral loading was applied to the head of the structural

column, which is supported by a 120 cm diameter pile. Based on this analysis, equivalent horizontal soil stiffness curves (p-y) were generated for various embedment depths of columns (or piles), corresponding to different construction stages of the top-down method. These curves were obtained at one-meter intervals along the embedded depth.

Figure A2.5 presents two examples of the equivalent lateral soil stiffness curves derived from these analyses for various construction stages at one-meter depth intervals. These curves, represented as force-displacement data, were incorporated into the structural modeling, analysis, and staged construction design within the ETABS structural software.

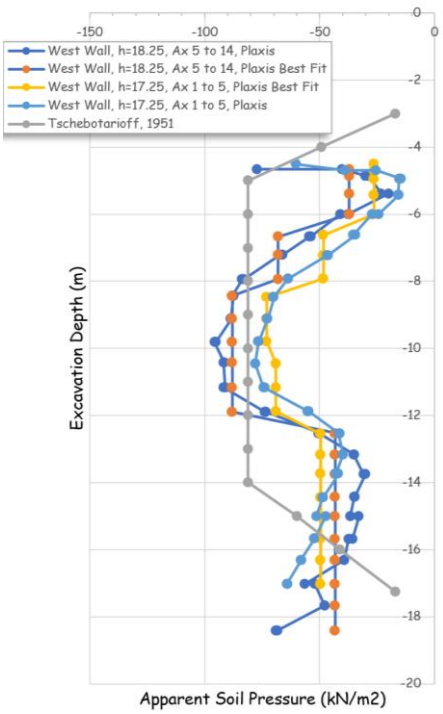


Figure A2.4 West side soil pressure distribution with a surcharge from the 8-story adjacent building

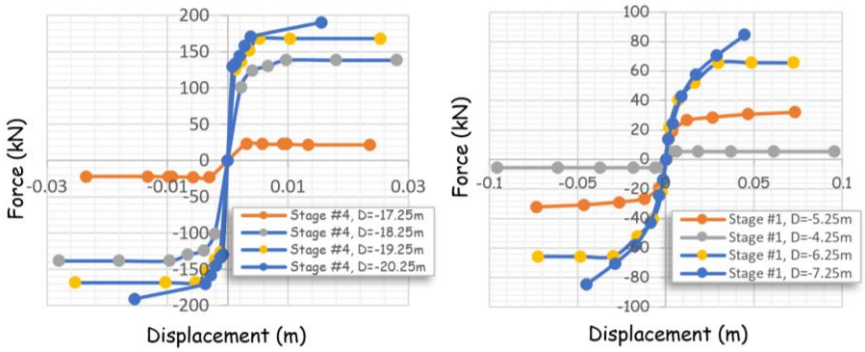


Figure A2.5 Force–displacement (p-y) curves representing the equivalent horizontal soil stiffness were obtained from numerical modeling for the column supported on the pile. The pile beneath the column was analyzed at various construction stages of the top-down method, with evaluations conducted at one-meter depth intervals along both the column and pile.

A2.2.4 Static Geotechnical Analysis of a Sample from the East-West Frame (Grid Line 5 of the Column Layout) for Staged Top-down Construction

In the numerical geotechnical modeling of structural frames in Plaxis 2D, the geotechnical properties of the project site's soil layers were assigned based on the geotechnical investigation report (Table A2.1). Figure A2.6 shows the structural column layout plan used as the basis for numerical modeling and Figure A2.3 illustrates the modeled geometry of the East-West structural frame along grid line 5, incorporating surcharge loads from adjacent buildings to the east and west. Figures A2.7 through A2.10 present the total deformation contour, horizontal displacements, vertical settlements, and induced deformations on neighboring structures, respectively. These results correspond to the construction stage in which excavation is completed and soil pressures are fully transferred to the structural frame and retaining walls. Excavation progresses in 1-meter increments below each slab level before proceeding to the next basement level. In the modeling, retaining walls are considered completed 1 meter below each newly cast slab. Since the actual excavation will follow

an alternate panel ("checkerboard") sequencing while the analysis assumes 2D plane-strain conditions, the predicted deformations are conservatively higher than those expected in the field.

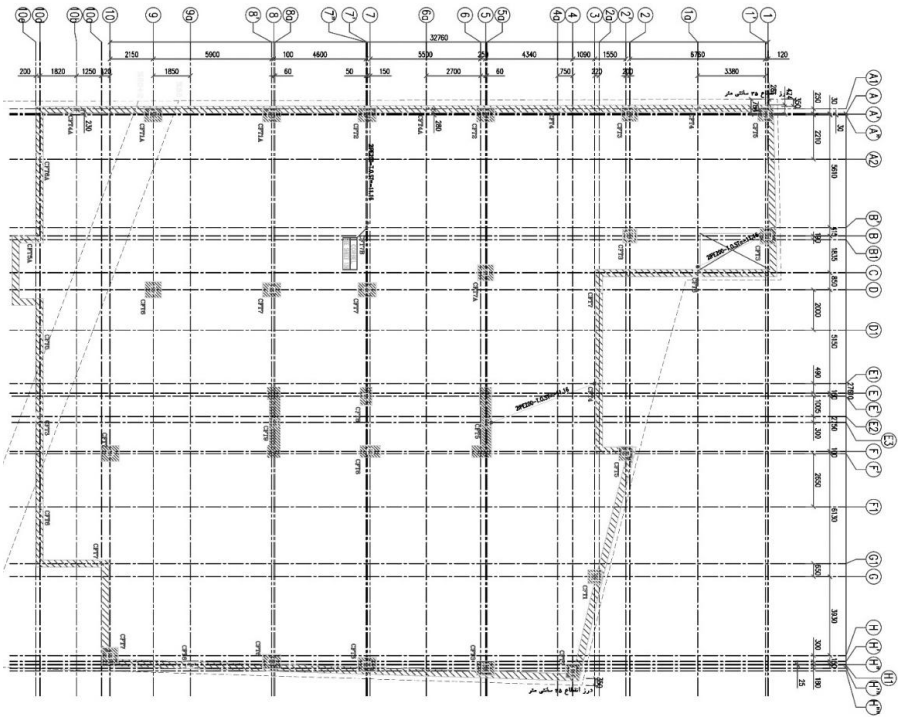


Figure A2.6 Column layout of the concrete structure, showing basement levels with CFT columns

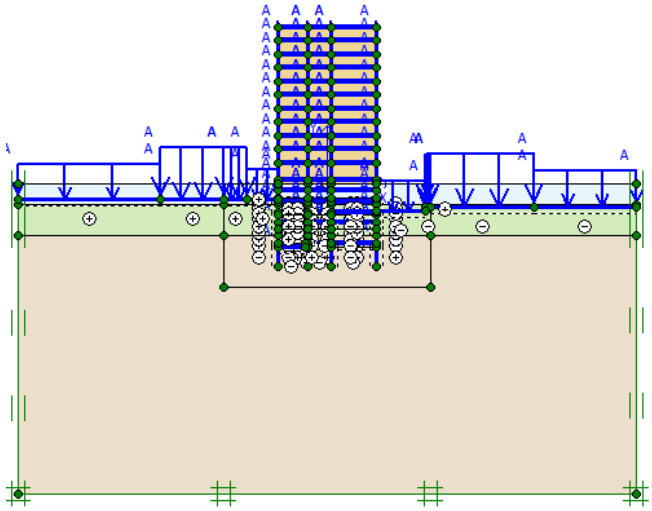


Figure A2.7 Geometrical model of the East-West frame

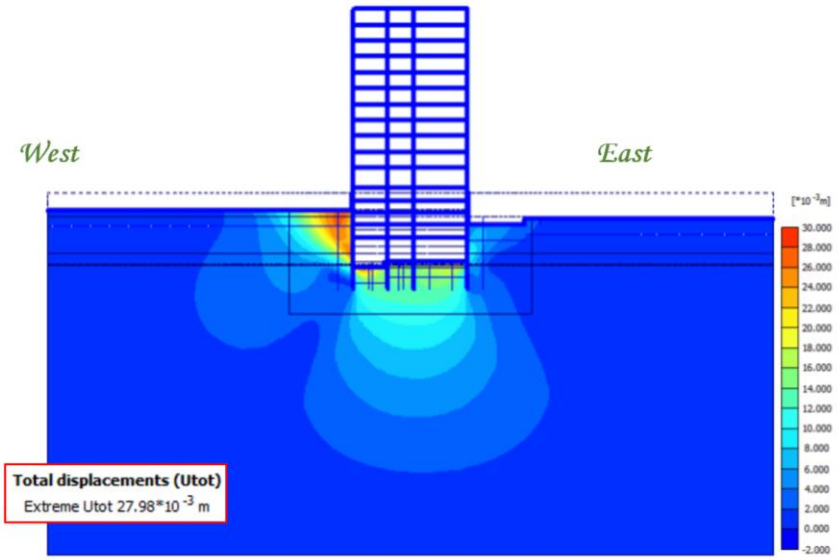


Figure A2.8 Excavation-induced deformation contour of the completed top-down structure at the end of the excavation operations along grid line 5 of the East-West frame

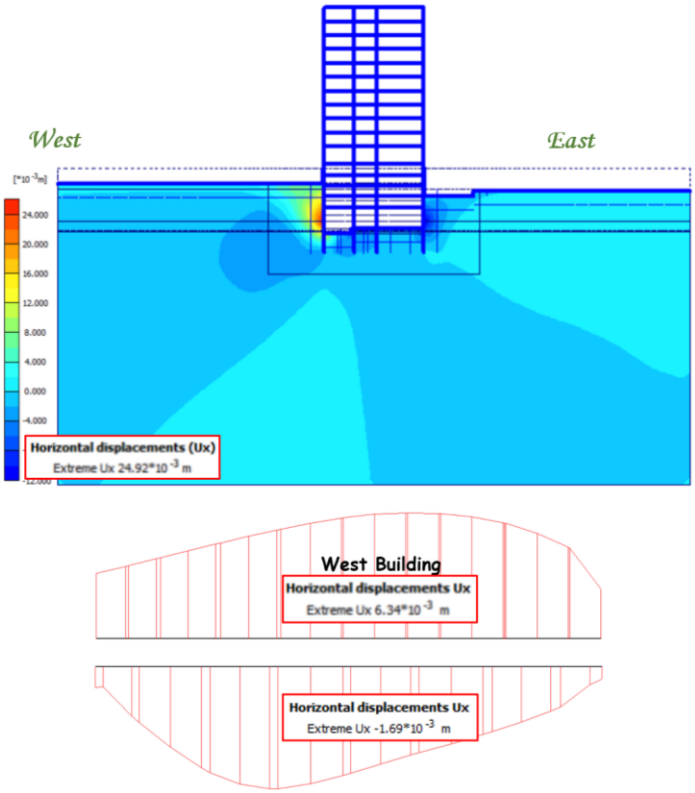


Figure A2.9 Horizontal excavation-induced displacement contour of the completed top-down structure at the end of the excavation operations along grid line 5 of the East-West frame

Figure A2.11 illustrates the results of the phi/c reduction analysis used to assess the safety factor for excavation stability. The shear force, axial force, and bending moment diagrams for the structural members were examined as well. Figure A2.12 provides an example of these diagrams for the piles supporting the structure's columns. This analysis served as an additional verification step for the structural design, while the primary design of the structural members was carried out using structural design software.

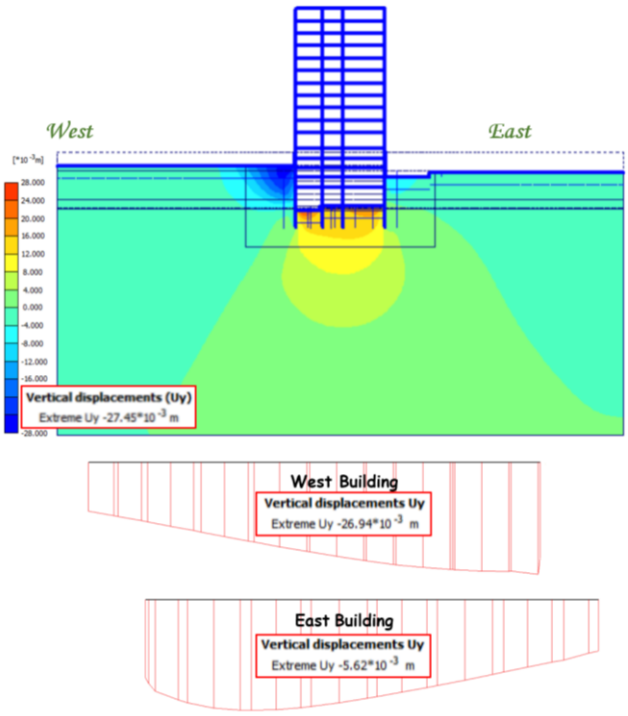


Figure A2.10 Excavation-induced settlement contour of the completed top-down structure at the end of the excavation operations along grid line 5 of the East-West frame

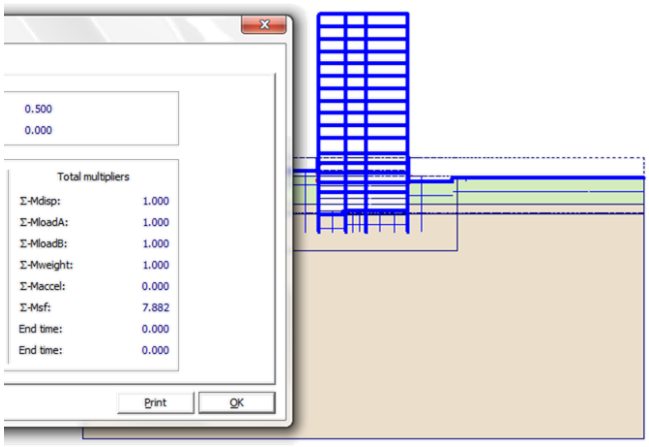


Figure A2.11 Global stability safety factor (FS = 7.9) after completing the excavation and constructing the structure using the top-down method (East-West frame, grid line 5).

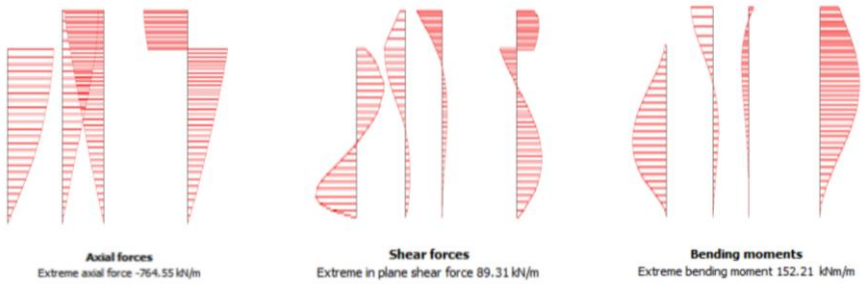


Figure A2.12 Envelope diagrams for axial forces, shear forces, and bending moments along the pile length (East-West frame, grid line 5)

A2.3 Structural Modeling and Monitoring of Top-down Construction Stages

Using the results of geotechnical data and analyses, the designed structure was modeled, analyzed, and designed for ultimate loads in ETABS at each construction stage of top-down construction. The adequate performance of structural components was verified through the definition of nonlinear staged construction loads in ETABS, following the previously outlined construction phases.

Since the structural columns were embedded in the soil during pre-excavation stages and exhibited pile-like behavior, appropriate boundary conditions were established using geotechnical analysis results and equivalent soil stiffness data to accurately simulate the construction process. In the numerical structural model, equivalent soil springs were assigned around embedded columns for each stage. As excavation progressed and retaining walls were constructed, these springs were modified or removed to reflect the updated embedment depth of columns and piles at lower elevations. This process continued until the foundation level was reached.

The modeling of construction stages also included isolated piles beneath the columns and the foundation to ensure proper stress distribution and load transfer during top-down execution. In the structural model, columns supported by piles were fully

incorporated from the beginning, followed by the sequential construction of basement levels and the simultaneous erection of two superstructure floors.

Figure A2.13 illustrates the overall structural model with equivalent soil springs around embedded columns in the top-down method, where each spring represents the horizontal soil stiffness surrounding the column. An additional spring—based on pile bearing capacity analysis—was placed beneath the pile tip to account for axial stiffness, complementing the horizontal soil springs. Figure A2.14 presents the top-down construction staging model in ETABS.

Accounting for the project's geotechnical conditions, execution constraints, and the bearing capacity of 120-cm-diameter piles beneath columns, iterative coordination between geotechnical and structural consultants confirmed that three superstructure floors can be constructed simultaneously with basement levels. This approach ensured controlled pile settlements within allowable limits while meeting the economic and technical requirements of the project.

The results confirmed the adequate performance of the structural components, particularly the steel core columns, which served as the primary load-bearing elements throughout staged construction. It is important to note that the load combinations considered for top-down construction phases included construction loads such as dead loads, earth pressures, and live loads during the construction process.

The axial forces acting on the piles and the settlements resulting from top-down construction are illustrated in Figures A2.15 and A2.16, respectively. The maximum predicted settlement was 2.5 cm for central columns, which were under high axial loads and were supported by bell-bottom piles. For peripheral columns, the maximum settlement was limited to 1.2 cm. These values were

monitored and verified during construction through instrumentation to ensure compliance with design predictions.

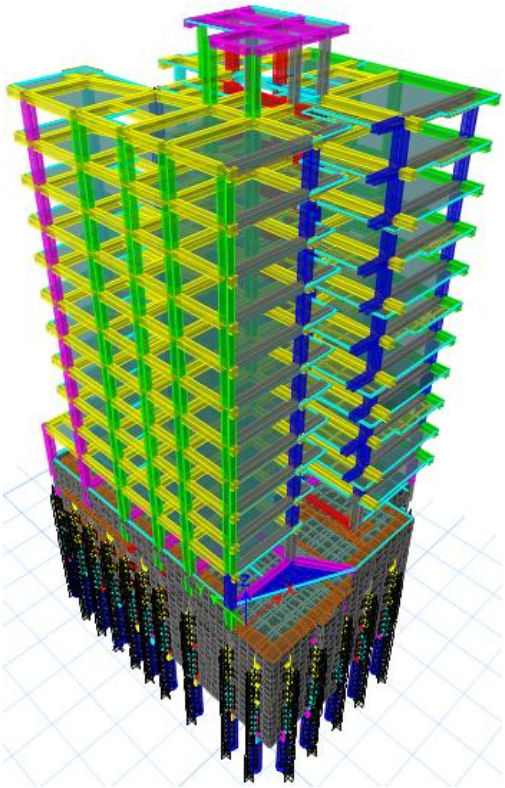


Figure A2.13 ETABS-produced overall model of the pile-supported structure, representing boundary conditions around columns

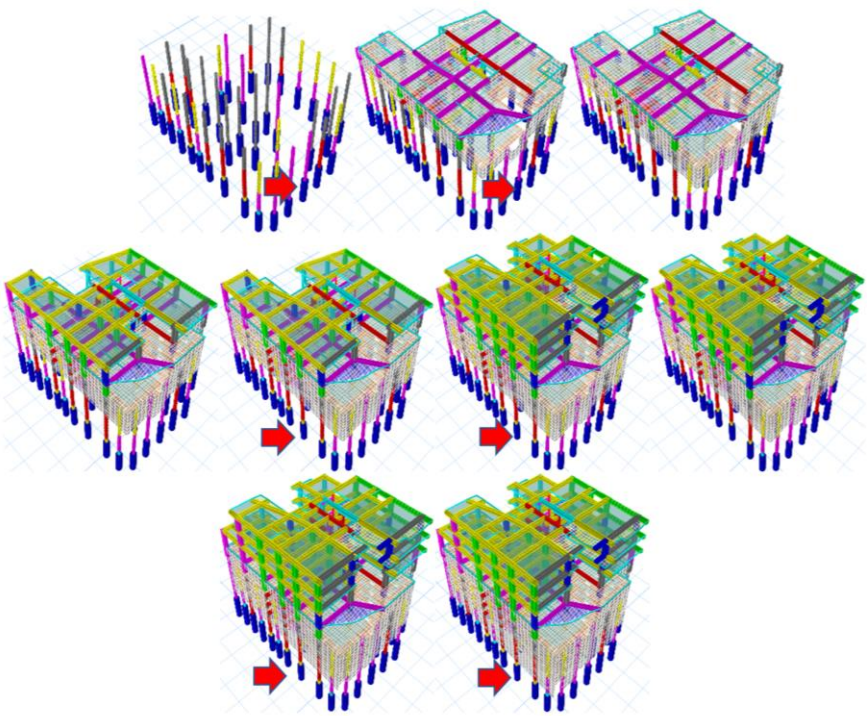


Figure A2.14 Structural stages based on boundary conditions produced by ETABS; the first stage represents the embedment of the pile-supported columns, and the final stage shows the completed basement floors as well the three superstructure floors

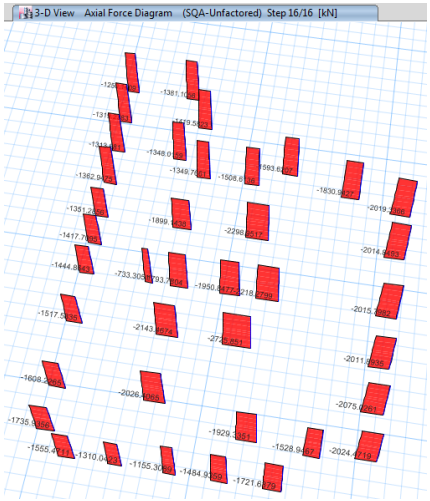


Figure A2.15 Final axial loads on piles supporting columns at the final stage of top-down structural analysis using unfactored design loads in ETABS

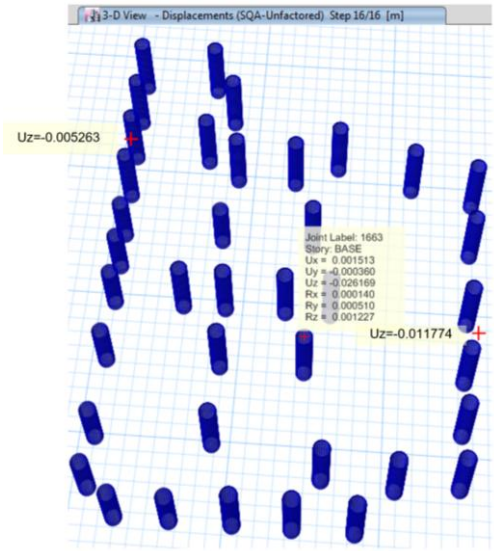


Figure A2.16 Maximum predictable settlement when the three superstructure floors and all basement floors are constructed under axial loads in piles supporting columns at the final stage of the top-down structural analysis using unfactored design loads in ETABS

About the Book

The Top-Down Construction Method offers a safe, efficient, and sustainable solution for complex underground and high-rise projects, particularly on geotechnically challenging or sensitive sites. Increasingly applied in Iran for tunnels, metro stations, and building, it requires integrated expertise across multiple engineering domains. Despite its technical depth, few comprehensive resources exist—this book fills that gap, combining theoretical foundations with practical guidance for engineers, consultants, and contractors.

About the Authors

This publication is the result of a coordinated scholarly effort involving a multidisciplinary team of leading Iranian academics and engineering professionals, each recognized for their substantial contributions to the development and application of the top-down construction method. Drawing on decades of research, field experience, and institutional knowledge, the authors have been at the forefront of adapting and advancing top-down construction practices in Iran.

Their collaborative authorship reflects the inherent complexity and interdisciplinary requirements of this construction technique, encompassing various fields of engineering such as geotechnical, structural, mechanical and electrical (M&E), surveying, structural health monitoring (SHM), instrumentation, and risk management.

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Together, this distinguished team presents a technically rigorous and practice-oriented reference tailored to the demands of today's top-down construction projects.

ISBN: 978-600-113-582-8

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