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Presented by:

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Subject

Numerical Modeling of Bridge Pier of Pile Group in Cohesive Soil

Presented and defended publicly, on 07/06/2023, in front of the jury composed of :

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بسم الله الرحمن الرحيم

Abstract

Numerical modeling plays a crucial role in the analysis and design of bridge piers founded on pile groups in cohesive soil. The FB-MultiPier program is used to analyze pier and pile systems, and it requires a comprehensive dataset as input including various information related to soil properties, pier specifications, pile details, and pier and pile cap information. This analysis program couples structural with static soil models for axial, lateral and torsional soil behavior to provide a robust system of analysis for bridge pier structures and foundation systems. FBMultiPier calculates the response of the bridge pier and pile group, providing output such as bending moments, shear forces, axial forces, and displacements and generating graphs as a result of the simulations, allowing us to analyze and interpret the findings. Based on the analysis results, the design is performed by adjusting parameters such as pile spacing, diameter, length, or shape of the pier. Iteratively analyzing and refining the design until the desired performance and safety criteria are achieved using the D/C Ratio.

The findings have practical implications for optimizing bridge design and developing efficient foundation systems, ultimately leading to safer and more reliable bridge structures.

ملخص

تلعب النمذجة العددية دوًرا حاسًما في تحليل وتصميم أرصفة الجسور التي تأسست على مجموعات الخوازيق في تربة متماسكة. يستخدم برنامجFB-MultiPier لتحليل أنظمة الرصيف والأكوام، ويتطلب مجموعة بيانات شاملة كمدخلات بما في ذلك المعلومات المختلفة المتعلقة بخصائص التربة، ومواصفات الرصيف، وتفاصيل الكومة، ومعلومات الرصيف وأغطية الكومة. يقرن برنامج التحليل هذا الهيكلية بنماذج التربة الثابتة لسلوك التربة المحوري والجانبي والالتوائي لتوفير نظام تحليل قوي لهياكل رصيف الجسر وأنظمة الأساس. يحسبFB-MultiPier استجابة رصيف الجسر ومجموعة الخوازيق، مما يوفر مخرجات مثل لحظات الانحناء وقوى القص والقوى المحورية والتهجير وتوليد الرسوم البيانية كنتيجة لعمليات المحاكاة، مما يسمح لنا بتحليل وتفسير النتائج. استنادًا إلى نتائج التحليل، يتم تنفيذ التصميم عن طريق ضبط المعلمات مثل تباعد الخوازيق، أو القطر، أو الطول، أو شكل الرصيف. تحليل التصميم وتنقيحه بشكل متكرر حتى يتم تحقيق معايير الأداء والسلامة المطلوبة باستخدام نسبةD / C.

النتائج لها آثار عملية لتحسين تصميم الجسر وتطويره بكفاءة أنظمة الأساس، مما يؤدي في النهاية إلى هياكل جسر أكثر أمانًا وموثوقية.

Dedicated

This modest work is dedicated to:

The dearest being of my life, my mother (Noudjoud)

The one who made me a woman, my father (Abd El-Kader)

My dear brothers (Oussama, Mouhamed, Ghaith)

To all my teachers who have supervised me throughout the years of my studies.

To my dear supervisor, Dr MESSAOUD Farid who believed in my work and patiently supported me.

I dedicate this work to all those who participated in my success.

BENTIBA Amani

Dedicated

I dedicate this project to God Almighty my creator, my strong pillar, my source of inspiration and understanding. Also to my dear father Salah, who has been nicely my supporter until my research was fully finished, and my beloved mother Mebarka who for months past, has encouraged me attentively with her fullest and truest attention. Then to my siblings, Djouhaina, Ikhlas, Djihad who have been my biggest cheerleaders and confidants.

To my dear supervisor, Dr. MESSAOUD Farid who believed in my work and patiently supported me.

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Chapter 1

Introduction to Bridges

1 History of Bridges

Bridge is a structure that provides passage over obstacles such as valleys, rough terrain or bodies of water by spanning those obstacles with natural or manmade materials. They first begun be used in ancient times when first modern civilizations started rising in the Mesopotamia.

In the beginning bridges were very simple structures that were built from easily accessible natural resources-wooden logs, stone and dirt. Because of that, they had ability only to span very close distances, and their structural integrity was not high because mortar was not yet invented and rain slowly but constantly dissolved dirt fillings of the bridge. Revolution in the bridge construction came in Ancient Rome whose engineers found that grinded out volcanic rocks can serve as an excellent material for making mortar.

Throughout the history of civilization bridges have been the icons of cities, regions, and countries. Bridges are necessary for civilization to exist, and many bridges are beautiful.. Modern bridge engineering has its roots in the nineteenth century, when wrought iron, steel, and reinforced concrete began to compete with timber, stone, and brick bridges as shown in Figure below:



Roman Bridge of Córdoba (Spain) Sidi M'Cid Bridge (Algeria) London Bridge (UK)

Figure1.1 Bridge Symbols of Ancient Cities.

By the beginning of World War II, the transportation infrastructure of Europe and North America was essentially complete, and it served to sustain civilization. The iconic bridge symbols of modern cities were in place (Figure 1.2): Golden Gate Bridge of San Francisco, Brooklyn Bridge, London Bridge, Eads Bridge of St. Louis, and the bridges of Paris, Lisbon, and the bridges on the Rhine and the had seven beautiful bridges across the Danube of Budapest. Then came World War II, and most bridges on the European continent were destroyed. A renaissance of bridge engineering started in Europe, then spreading to America, Japan, China, and Africa. The past 60 years of bridge engineering have brought new forms of bridge architecture (plate girder bridges, cable stayed bridges, segmental prestressed concrete bridges, composite bridges), and longer spans. Meanwhile enormous knowledge and experience have been amassed by the profession, and progress has benefitted greatly by the availability of the digital computer.

Hong Kong-Zhuhai-Macao

Sao Paulo city Bridge (Brazil)

Golden Gate Bridge (SF, USA)

Figure1.2 Bridge Symbols of Modern Cities.

2 Purpose of a bridge

The basic purpose of a bridge is to carry traffic over an opening or discontinuity in the landscape. Various types of bridge traffic can include pedestrians, vehicles, pipelines, cables, water, and trains, or a combination thereof. An opening can occur over a highway, a river, a valley, or any other type of physical obstacle. The need to carry traffic over such an opening defines the function of a bridge. The design of a bridge can only commence after its function has been properly defined. Therefore, the process of building a bridge is not initiated by the bridge engineer. Just like roads or a drainage system, or other types of infrastructure, a bridge is a part of a transportation system and a transportation system is a component of a city’s planning efforts or its area development plan. The function of a bridge must be defined in these master plans.

3 Bridge Types

All bridges in the world can be grouped into seven basic types: girder bridge, arch bridge, cable-stayed bridge, and suspension bridge (Figure 1.1). There are also varying possible combinations, such as the cable-stayed and suspension scheme proposed by Franz Dishinger, and the “partially cable-supported girder bridge” (Tang, 2007). For simplicity, we can drop the word “partially” in this name and call it cable-supported girder bridge. It is a combination of a girder bridge and any one of the aforementioned bridge types. The extra dosed bridge is a special subset of the cable-supported girder bridge.

3.1 Beam Bridges

A beam bridge, sometimes called a girder bridge, is a rigid structure that consists of one horizontal beam supported at each end, usually by some kind of pillar or pier. In structural terms, it is the simplest type of bridge and is a popular selection because of its inexpensive construction costs. It began as a felled log supported by opposing river banks that was used to span a river or other body of water.

This type of bridge works on the principles of compression and tension, so a strong beam is needed to resist twisting and bending under the weight it must support. When the bridge is loaded, by traffic, the beam bends which causes the top surface to be compressed and the bottom surface to be stretched or put in tension (Fgure 1.3).


Lake Pontchartrain Causeway Manchac Swamp Bridge

Figure1.3 Beam Bridge (Louisiana State)

3.2 Arch Bridges

An arch bridge is a type of architectural structure that relies on a curved, semi-circular shape for support. Arch bridges have abutments at each end. The weight of the bridge is thrust into the abutments at either side.

The earliest known arch bridges were built by the Greeks. These bridges uses arch as a main structural component (arch is always located below the bridge, never above it). Thousands of years ago, Romans built arches out of stone (Figure 1.4). Today, most arch bridges are made of steel or concrete, and they can span up to 800 feet (243.8 m) as shown in Figure 1.4. They are often chosen for their strength and appearance.


Arch Bridge with Brick and Stone Arch Bridge with Concrete.

Figure1.4 Arch Bridges

3.3 Cantilever Bridges

Cantilever bridges are based on structures that project horizontally into space, supported at only one end - like a spring board. For small footbridges, the cantilevers may be simple beams; however, large cantilever bridges designed to handle road or rail traffic use trusses built from structural steel, or box girders built from prestressed concrete. This type of bridge has been used for pedestrians, trains, and motor vehicles. Cantilevers are especially useful for spanning a waterway without dividing it with river piers (Figure 1.5).

Forth Bridge (UK) Gate Bridge (Tokyo)

Figure1.5 Cantilever Bridges

3.4 Truss Bridges

Truss Bridges are structures built up by jointing together lengths of material to form an open framework - based mainly on triangles because of their rigidity. They are very strong and can support heavy loads (Figure 1.6).

One bridge historian describes a truss bridge in this manner: "A truss is simply an interconnected framework of beams that holds something up. In a truss bridge, two long - usually straight members known as chords - form the top and bottom; they are connected by a web of vertical posts and diagonals. The truss does not support the roadway from above, like a suspension bridge, or from below, like an arch bridge; rather, it makes the roadway stiffer and stronger, helping it hold together against the various loads it encounters." (Eric DeLony, The Golden Age of the iron Bridge, Invention and Technology, 1994).

Jennings Randolph Bridge (WV, USA) Kamagari Bridge (Japan)

Figure1.6 Truss Bridges

3.5 Suspension Bridges

Suspension bridge is a specific type of bridge where the platform, usually a road, is hung below small vertical cables called suspenders. Smaller vertical suspender cables are attached to the main cables to support the deck below.

Suspension bridges. as seen in Figure 1.7, are built so that the compression forces from the weight applied to the deck are transferred through the suspenders to the large cables running between the towers. The force then follows the cables through the towers and is passed into the ground.

Golden Gate Bridge (SF, USA) 1915 Canakkale Bridge(Turkey)

Figure1.7 Suspension Bridges

3.6 Cable-Stayed Bridges

A cable-stayed bridge is a structure with several points in each span between the towers supported upward in a slanting direction with inclined cables and consists of main tower(s), cable-stays, and main girders.

The cables at the towers can be arranged in parallel (harp), fan, star, or mixed configuration. Various structural solutions are used for the towers: single pylons, double-leg portals (vertical, slightly angled, free-standing, or interconnected as a portal frame, with “A,” “H,” “Y,” or inverted “Y” shaped arches (Figure 1.8).

Figure1.8 Type of Cable-Stayed Bridges

4 Types of Bridges by Mobility

4.1 Fixed Bridges

Fixed bridges are pretty simple: They’re anchored in place and meant to provide a steady, secure passage across a river, canyon, rail line, roadway, or other obstacles. Many of the bridge styles described above are in this category.

I-beam girders can provide stability in beam bridges, such as highway overpasses, but box girders — enclosed tubes, usually rectangular — provide better protection against torsion.

4.2 Temporary Bridges

Pontoon bridges are most commonly used in wartime to transport troops, supplies, and military vehicles. These are generally temporary structures that float directly on the water atop pontoons - containers filled with air to provide buoyancy.

They are a successor to the ancient practice of lining up ships or rafts end-to-end to form a bridge. One drawback of pontoon bridges is that, because they rest directly on the water’s surface, they obstruct any watercraft trying to navigate the channel they cross.

4.3 Movable Bridges

The category of movable bridges includes lifting and swinging bridges. These allow a portion of the structure to move out of the way and allow passage of tall ships in a waterway. Here are some examples:

A vertical lift bridge is a kind of truss bridge that's raised using cables affixed to the deck. These allow it to be raised via pulleys attached to the top of a tower or pylon on either side of the waterway. The deck remains horizontal as it's raised, and its maximum height is dictated by the height of the towers.

The deck on a bascule bridge, or drawbridge, by contrast, is lifted from the base, like a door swinging upward, often by hydraulics.

Swing bridges also allow watercraft to pass, but using a different method: They rotate horizontally, or swing like an opening door, on a pedestal. They are not as common, but they are used occasionally in places too wide for a lift bridge.

5 Common Types of Bridges by Function

5.1 Aqueduct/Viaduct Bridge

An aqueduct is a "Water Bridge" in Latin. The Romans used arched aqueducts to carry water from one place to another. A viaduct is a roadway elevated by a series of arches over an extended distance.

5.2 Culvert

Culverts aren't technically bridges, but they're similar. Simple structures usually surrounded by soil or other fill, these allow water to flow underneath rather than across a road, trail, or rail line. They're often made of concrete, but some are simple corrugated pipes.

5.3 Double-Decked Bridge

A double-decked bridge can accommodate more traffic in densely populated areas. The Yangsigang Yangtze River Bridge in China, which opened in 2019 at a cost of $1.27 billion, is the longest double-decker suspension bridge in the world, at 5,500 feet.

Its upper deck features six lanes for each direction of freeway traffic, plus pedestrian footpaths and sightseeing areas to the sides. The lower deck carries another four local motor vehicle lanes, two lanes for non-motorized vehicles, and two more pedestrian walkways.

5.4 Pedestrian Bridge

Footbridges, or pedestrian bridges, can be simple spans across canyons, streams, or roadways wide enough for a person (or two people side-by-side) to cross. The earliest bridges were stepping stones or fallen trees. This type also includes swinging bridges and boardwalks, which typically traverse lower, marshy, or sandy land.

5.5 Pipeline Bridge

A pipeline bridge, as you might expect, is built to carry a gas- or liquid-bearing pipeline. These run through places where it isn't possible to build the pipeline under a river or other obstacle. These are often suspension bridges.

5.6 Train Bridge

Truss bridges became prominent in the 1800s, when railroads were the pinnacle of transportation, so it's no surprise that many truss bridges carry rail lines. Another kind of train bridge is the trestle, which consists of multiple short beams end-to-end, supported by (often wooden) frames placed close together to cross a long span.

5.7 Vehicle Traffic Bridge

A traffic bridge is wide and sturdy enough for at least one vehicle to traverse in a single direction, although usually it accommodates at least two lanes of opposing traffic.

6 Types of Bridge Materials

Around the world, bridges are made of almost any material at hand, including ropes, vines, even trash. The following materials are the most common used in bridge building:

6.1 Wood

A popular material for trusses and trestles in the 1800s, wood was also used for covered bridges. Its use gave way to more durable options that weren't susceptible to warping, splintering, and termites.

6.2 Stone

A low-maintenance and durable option often used for arched bridges, stone was often used for bridge-building in the Roman era.

6.3 Concrete and Steel

In more modern times, a combination of concrete and steel is most often used for freeway overpasses, etc.

6.4 Advanced Materials

Construction materials are evolving to respond to specific environmental conditions and cut down on maintenance. These include fiber-reinforced plastics, high-performance concrete, and composite materials.

7 Forces that influence Different Bridge Designs

What determines the type of bridge that gets built in a particular place? Bridge designs are based on more than aesthetics. Some of the forces of nature that act upon bridges are:

7.1 Gravity

The downward pull is a bigger deal with bridges than buildings. Unlike a home or a skyscraper, most of what's under a bridge is empty space.

7.2 Load

The weight of the bridge itself is combined with the weight of whatever it carries. The longer a bridge is, and the more people, cars, and other things it carries, the heavier its load.

7.3 Compression

The pushing or squeezing force that creates inward movement toward the center, compression is what helps keep arched bridges standing. But with too much compression, a bridge can buckle.

7.4 Tension

In the opposite direction, tension is the pulling or stretching force that creates outward movement away from the center. Tension in vertical cables is what sustains suspension bridges. But with too much tension, a bridge can snap.

7.5 Torsion

This twisting force, often caused by environmental forces like wind, can cause dangerous movement in structures like suspension bridges. If the surface of a bridge twists enough while travelers are on it, they can be thrown off.

7.6 Shear

While torsion creates stress on a vertical plane, shear is a force that imparts a similar effect, but on a horizontal plane. It happens when environmental forces put pressure in opposite directions on a single fastened part of a bridge.

7.7 Vibration/ Resonance

When wind or movement across a bridge matches its natural frequency of vibration, it can cause a phenomenon called resonance. If vibrations are extreme enough, they can disrupt crossings and cause a collapse.

The interplay of these forces looks like this: Gravity pulls down on a bridge's structure. Forces of compression push the load inward onto piers at the middle of the bridge. The force of tension pulls the load outward onto abutments at both ends of the bridge.

When these forces act in opposite directions on a part of the bridge, it can create damage from shear. Wind and heavy loads can create torsion or vibration/resonance on a bridge, both of which also can be dangerous. Modern bridges are engineered to counteract these potentially threatening conditions.

8 Bridge Components and Elements

Every bridge can be divided broadly into three main parts: Superstructure/ Substructure/ Foundation (Figure 1.9 to 1.11).

8.1 Superstructure

Superstructure that part of the structure which supports traffic and includes deck, slab and girders. All the parts of the bridge which is mounted on a supporting system can be classified as a superstructure.

8.2 Substructure

Substructure that part of the structure, ie piers and abutments, which supports the superstructure and which transfers the structural load to the foundations.

8.3 Foundation

Foundation is the component which transfers loads from the substructure to the bearing strata. Depending on the geotechnical properties of the bearing strata, shallow or deep foundations are adopted. Usually, piles and well foundations are adopted for bridge foundations.

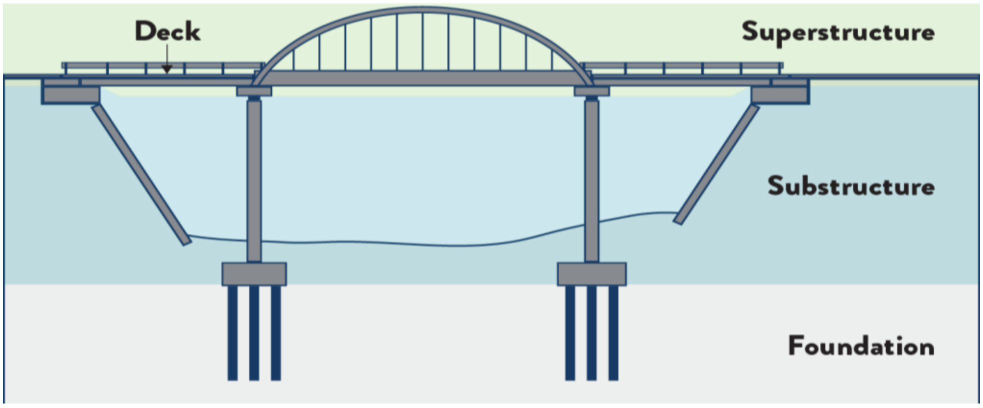


Figure1.9 Main Bridge PartsMain Bridge Parts

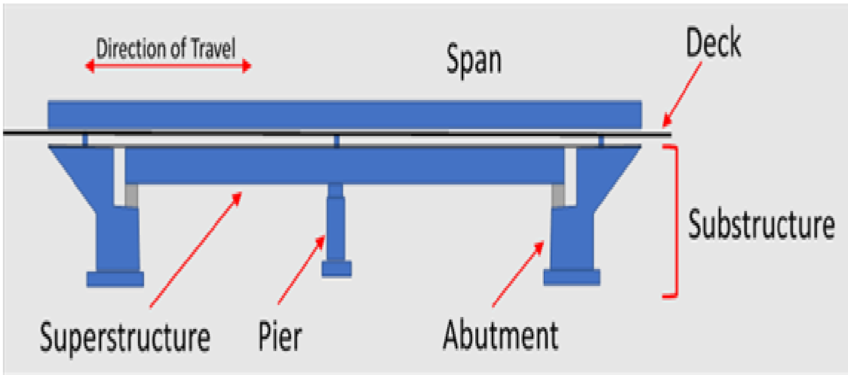


Figure1.10 Major Bridge Components

The various parts and components of a bridge are as follows:

Span : the distance between two bridge supports, whether they are columns, towers or the wall of a canyon.

Deck : a bridge floor directly carrying traffic loads. Deck transfers loads to the Girders depending on the decking material.

Beam : a rigid, usually horizontal, structural element.

Beam / Girder : is that part of superstructure which is under bending along the span. It is the load bearing part which supports the deck.

Bearing : Bearing transfers loads from the girders to the pier caps.

Pier : a vertical supporting structure, such as a pillar.

Pier Cap: the component which transfers loads from the superstructure to the piers. Pier cap provide sufficient seating for the Bridge girders.

Cantilever : a projecting structure supported only at one end, like a shelf bracket or a diving board.

Truss : a rigid frame composed of short, straight pieces joined to form a series of triangles or other stable shapes.

Pile Cap and Piles : Pile foundation is the most commonly used foundation system for bridges. Pile is a slender compression member driven into or formed in the ground to resist loads. A reinforced concrete mass cast around the head of a group of piles to ensure they act together and distribute the load among them it is known as pile cap.

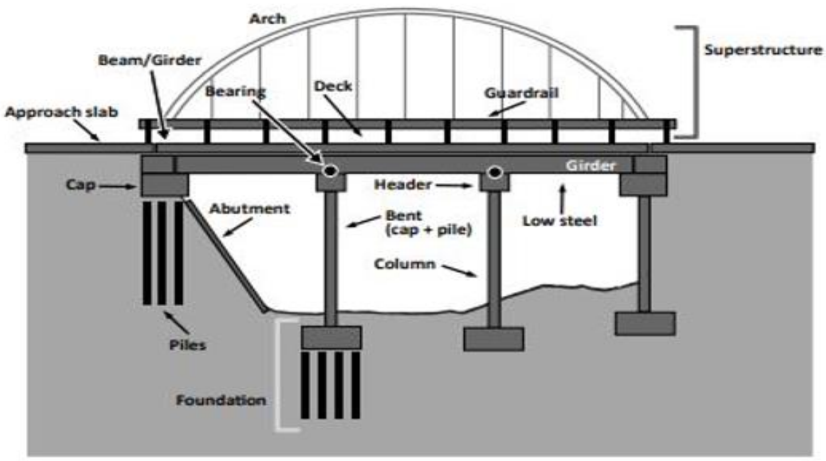


Figure1.11 Bridge elements

Chapter 2

Bridge Piers

1 Introduction

A bridge pier is a type of structure that extends to the ground below or into the water. It is used to support bridge superstructure and transfer the loads to the foundation. The bridge pier can be constructed to be substantially attractive and strong in order to withstand both vertical and horizontal loads. It also does not hinder water flow or tide if the bridge spans the water.

Bridge piers may be built using concrete, stone, or metal. Concrete is commonly specified as construction materials provided that the pier is submerged in water since metal is prone to rust in water. It is constructed in many locations like waterways, or dry lands on which highway systems are built as overpasses.

Piers provide vertical supports for spans at intermediate points and perform two main functions: transferring superstructure vertical loads to the foundations and resisting horizontal forces acting on the bridge. Although piers are traditionally designed to resist vertical loads, it is becoming more and more common to design piers to resist high lateral loads caused by seismic events. Piers are predominantly constructed using reinforced concrete. Steel, to a lesser degree, is also used for piers. Steel tubes filled with concrete (composite) columns have recently gained more attention.

2 Structural Types

Pier is usually used as a general term for any type of intermediate substructures located between horizontal spans and foundations. However, from time to time, it is also used particularly for a solid wall in order to distinguish it from columns or bents.

There are several ways of defining pier types. They vary according to structural design, aesthetic and economic factors. The variables that go into the design of the piers are therefore multiple: The size of the loads they receive from the deck, the height, the width of the deck, and the context in which they are located. One is by its structural connectivity to the superstructure: monolithic or cantilevered. Another is by its sectional shape: solid or hollow; round, octagonal, hexagonal, or rectangular. It can also be distinguished by its framing configuration: single- or multiple- column bent; hammerhead or pier wall. Figure 2.1 shows a series of columns in a typical urban interchange. The smooth monolithic construction not only creates an esthetically appealing structure but also provides an integral system to resist the seismic forces. Figure 2.2 shows one example of water crossings, the newly constructed Skyway of San Francisco–Oakland Bay Bridge.

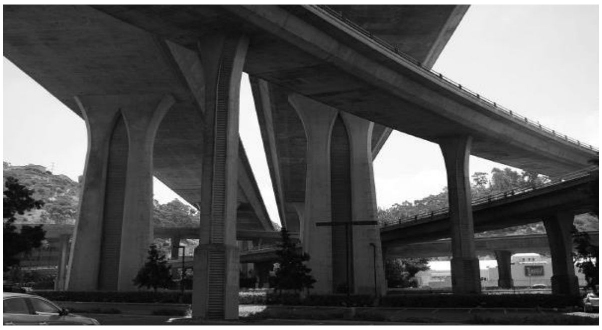


Figure 2.1 Columns in a typical urban interchange.

Figure 2.2 Columns in Skyway structure of SF–Oakland Bay Bridge.

2.1 Types of Bridge Piers

Piers are categorized into two major types based on its structure which include solid piers and open piers. These types are further classified into several types

2.1.1 Solid Piers

Solid piers possess solid and impermeable structure, and usually constructed from bricks, stone Masonry, mass concrete or reinforced concrete. Solid piers are categorized into solid masonry piers and solid reinforced concrete piers. Typical solid pier is shown in Figure 2.3.

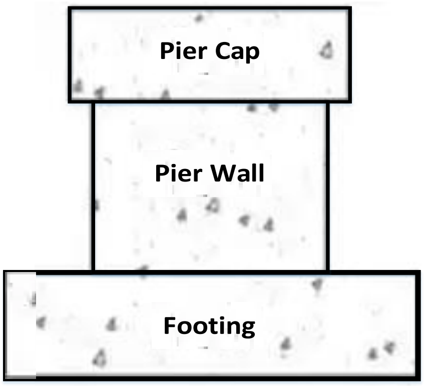


Figure 2.3 Typical Solid Pier

2.1.1.1. Solid Masonry Piers

The piers which are constructed with brick masonry, stone masonry, concrete etc. are known as solid masonry piers (Figure 2.4). Sometimes it is seen that in solid masonry piles the outer portion is constructed with the stone masonry and the inner part is filled with the help of mass concrete. In this way, it can save the cost of construction.

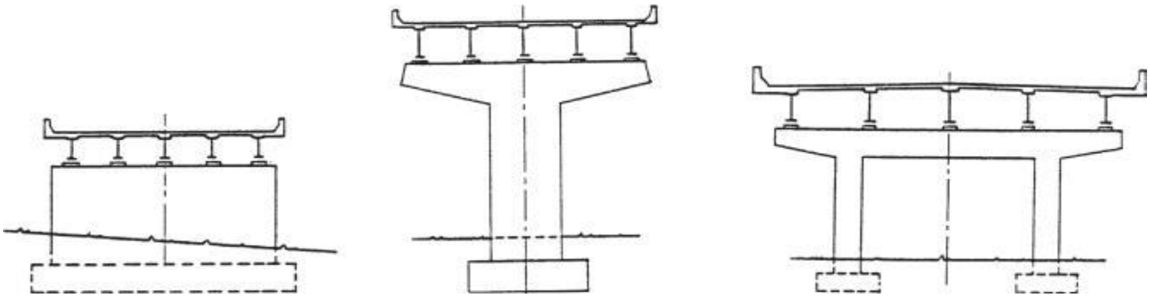
Figure 2.4 Solid Masonry Piers

2.1.1.2. Solid Reinforced Concrete Piers

Solid reinforced concrete piers as seen in Figure 2.5 are mostly constructed from reinforced concrete and commonly rectangular in cross-section. It is used in the case where the height of the piers is more and the solid masonry piers would not be strong enough to bear the load and can be uneconomical.

Figure 2.5 Solid Reinforced Concrete Piers

Solid wall piers, as shown in Figures 2.6a and 2.7, are often used at water crossings because they can be constructed to proportions that both are slender and streamlined. These features lend themselves well for providing minimal resistance to water flows.

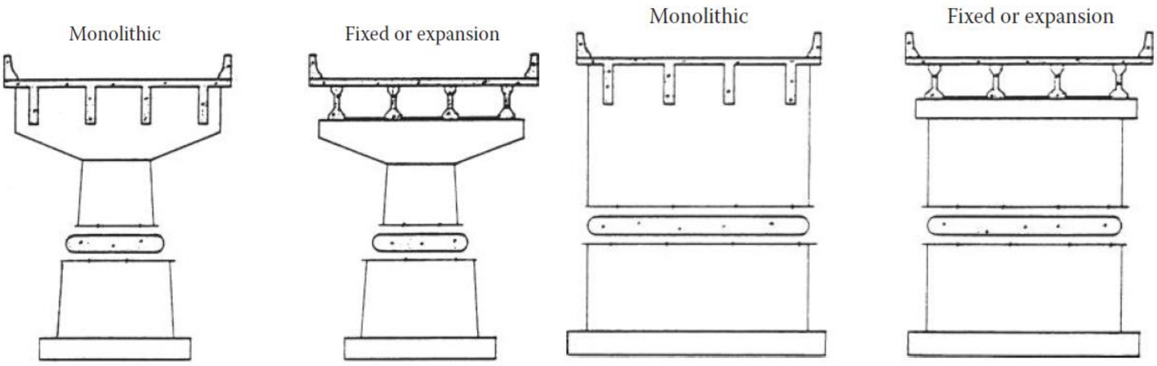


(a) Solid wall pier (b) Hammerhead pier (c) Rigid frame pier.

Figure 2.6 Typical pier types for steel bridges

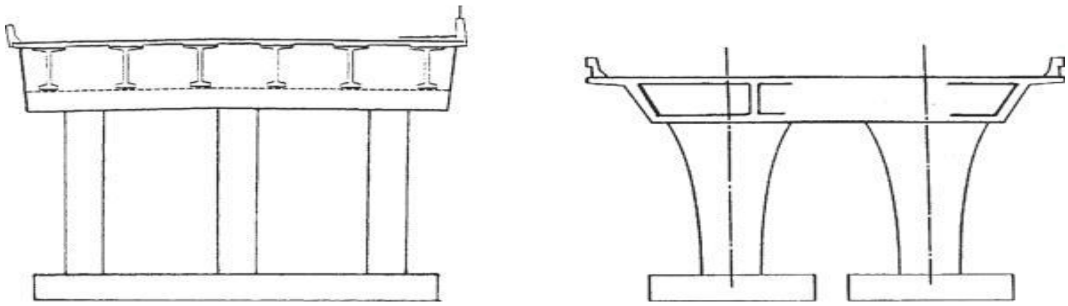
Hammerhead piers, as shown in Figure 2.6b, are often found in urban areas where space limitation is a concern. They are used to support steel girder or precast prestressed concrete girder superstructures. They are esthetically appealing and generally occupy less space, thereby providing more room for the traffic underneath.

A bent consists of a cap beam and supporting columns forming a frame. Bents, as shown in Figure 2.6c and Figure 2.8, can be used either to support a steel girder superstructure or as an integral bent where the cast-in-place construction technique is used. The columns can be either circular or polygonal in cross section. They are by far the most popular forms of piers in the modern highway systems.



(a) Hammerhead (b) Solid wall.

Figure 2.7 Typical pier types and shapes for river and waterway crossings



(a) Bent for precast girders (b) Bent for cast-in place griders.

Figure 2.8 Typical pier types for concrete bridges

2.1.2 Open Piers

Open piers permit the passage of water through the structure and classified into the following types

2.1.2.1. Cylindrical Piers

Cylindrical pier as shown in Figure 2.9 is constructed from cast irons or mild steel cylinders which are filled with concrete. This type of pier is suitable for bridges with moderate height. In certain cases, horizontal and diagonal steel bracing may be used to improve stability.

Figure 2.9 Cylindrical Piers.

2.1.2.2. Column Piers or Column Bent

This type of piers is suitable for bridge with significant height. It consists of a cap beam and supporting columns forming a frame (Figuree 2.10). Column bent piers can either be used to support a steel girder superstructure or be used as an integral pier where the cast-in-place construction technique is used. The columns can be either circular or rectangular in cross section. They are by far the most popular forms of piers in the modern highway system.

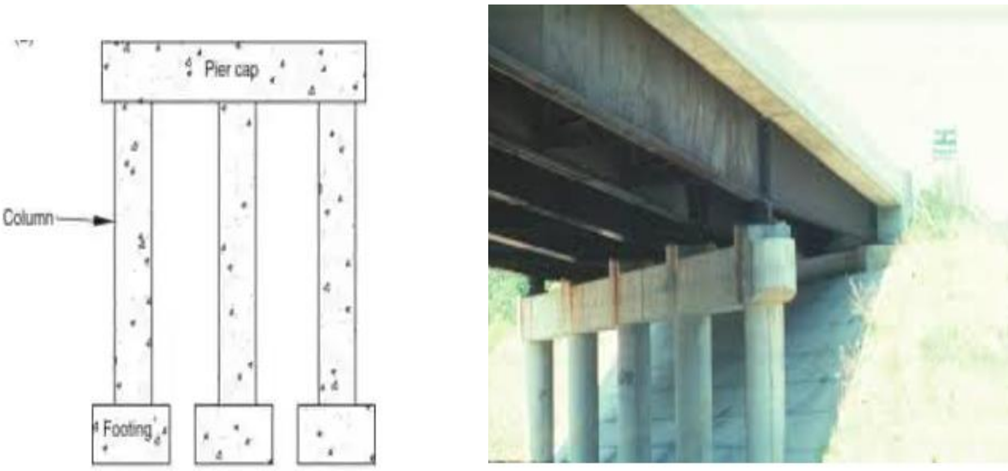


Figure 2.10 Column Bent Piers.

2.1.2.3. Multicolumn or Pile Bent

Multicolumn or pile bent or frame bent piers are composed of two or more column that supports a cap (Figure 2.11). Isolating footing is used for this type of piers if the spacing between columns are large otherwise combined footing would be more suitable. There is a problem of debris collection when the water is allowed to flow between the columns.

Figure 2.11 Pile Bent Pier

2.1.2.4. Pile Pier

Pile pier is the modification of multicolumn bent and used for the type of bent on low height and short span structure. So, pile pier or pile bents are specified when the ground is unstable and the low piers are required. Figure 2.12.

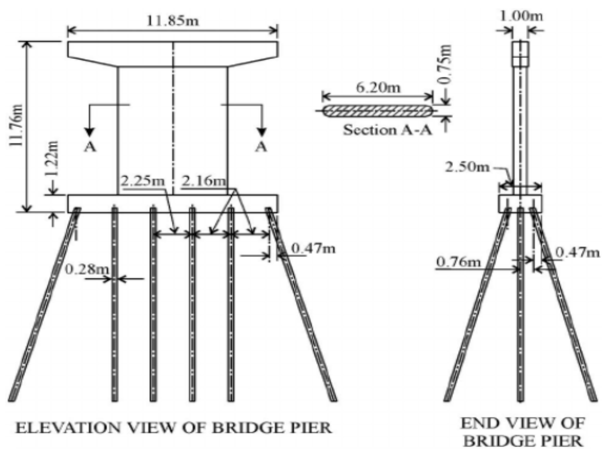


Figure 2.12 Pile pier

2.1.2.5. Trestle Pier or Trestle Bent

Trestle pier is composed of column with bent cap at the top. It is suitable for bridges in locations where river bed is firm and water current is slow. It is also employed for flyovers and elevated roads. Figure 2.13.

Figure 2.13 Trestle Pier or Trestle Bent

3 Design Loads

Piers are commonly subjected to forces and loads transmitted from the superstructure and forces acting directly on the substructure. Some of the loads and forces to be resisted by piers include the following:

- Dead loads
- Live loads and impact from the superstructure
- Wind loads on the structure and the live loads
- Centrifugal force from the live loads

- Drag forces due to the friction at bearings
- Stream flow pressure
- Ice pressure
- Earthquake forces
- Thermal and shrinkage forces
- Ship impact forces
- Force due to prestressing of superstructure
- Forces due to differential settlement of foundations

4 Design Criteria

Like the design of any structural component, the design of a pier or column is performed to fulfill strength and serviceability requirements. A pier should be designed to withstand the overturning, sliding forces applied from superstructure as well as the forces applied to substructures. A pier as a structure component is subjected to combined forces of axial, bending, and shear.

For a pier, the bending strength is dependent upon the axial force. In the plastic hinge zone of a pier, the shear strength is also influenced by bending. To complicate the behavior even more, the bending moment will be magnified by the axial force due to the P-Δ effect.

In current design practice, the bridge designers are becoming increasingly aware of the adverse effects of earthquake. Therefore, ductility consideration has become a very important factor for bridge design. Failure due to scouring is also a common cause of failure of bridges. In order to prevent this type of failure, the bridge designers need to work closely with the hydraulic engineers to determine adequate depths for the piers and provide proper protection measures.

Chapter 3

Deep Foundations

1 Introduction

A bridge foundation is part of the bridge substructure connecting the bridge to the ground. A foundation consists of man-made structural elements that are constructed either on top of or within existing geological materials. The function of a foundation is to provide support for the bridge and transfer loads or energy between the bridge structure and the ground.

A deep foundation is a type of foundation that the embedment is larger than its maximum plane dimension. The foundation is designed to be supported on deeper geologic materials because either the soil or rock near the ground surface is not competent enough to take the design loads, or it is more economical to do so.

Deep foundations are generally needed where the axial compression, axial tension, lateral load demand or a combination of the above cannot be satisfied by the near surface soil conditions. However, deep foundations should not be used indiscriminately for all subsurface conditions and for all structures. There are subsurface conditions where a driven pile, drilled shaft, micropile may be very difficult or costly to install. Ground improvement techniques can also be used with deep foundations as an economical means to improve lateral resistance in weak surficial soils (Rollins and Brown 2011).

2 Establishment of Deep Foundation

The first difficult problem facing the foundation designer is to establish whether or not the site conditions dictate that a deep foundation must be used. Vesic (1977) summarized typical situations in which piles may be needed. These typical situations as well as additional uses of deep foundations are shown in Figure 3.1.

Figure 3.1(a) shows the most common case in which the upper soil strata are too compressible or too weak to support heavy vertical loads. In this case, deep foundations transfer loads to a deeper competent stratum and act as predominantly toe bearing foundations. In the absence of a competent stratum within a reasonable depth, the loads must be gradually transferred, mainly through soil resistance along the shaft, Figure 3.1(b). An important point to remember is that deep foundations transfer load through unsuitable layers to suitable layers. The foundation designer must define at what depth suitable soil layers begin in the soil profile.

Deep foundations are frequently needed because of the relative inability of shallow footings to resist inclined, lateral, or uplift loads and overturning moments or excessive deformations. Deep foundations resist uplift loads by shaft resistance, Figure 3.1(c). Lateral loads are resisted either by vertical deep foundations in bending, Figure 3.1(d), or by groups of vertical and battered piles, which combine the axial and lateral resistances of all piles in the group, Figure 3.1(e). Lateral loads from overhead highway signs and noise walls may also be resisted by groups of deep foundations, Figure 3.1(f).

Deep foundations are often required when scour around footings could cause loss of bearing capacity at shallow depths, Figure 3.1(g). In this case the deep foundations must extend below the depth of scour and develop their full nominal resistance in the support zone below the level of expected scour. FHWA (Fedeal Highways Administrattion) scour guidelines using the Hydraulics Engineering Circular No. 18 (Arneson et al. 2012) require the geotechnical analysis of bridge foundations to be performed on the basis that all stream bed materials in the scour prism have been removed and are not available for bearing or lateral support. Costly damage and the need for future underpinning can be avoided by properly designing for scour conditions.

Liquefaction and other seismic effects on deep foundation performance must be considered for deep foundations in seismic areas. Soils subject to liquefaction in a seismic event may dictate that a deep foundation be used, Figure 3.1(h). Seismic events can also induce significant lateral loads to deep foundations. During a seismic event, liquefaction susceptible soils offer less lateral resistance, reduced shaft resistance, and can add drag load to a deep foundation.

Deep foundations are often used as fender systems to protect bridge piers from vessel impact, Figure 3.1(i). Fender system sizes and group configurations vary depending upon the magnitude of vessel impact forces to be resisted. In some cases, vessel impact loads must be resisted by the bridge pier foundation elements. Single deep foundations may also be used to support navigation aids.

In urban areas, deep foundations may occasionally be needed to support structures adjacent to locations where future excavations are planned or could occur, as in Figure 3.1(j). Use of shallow foundations in these situations could require future underpinning in conjunction with adjacent construction.

Deep foundations are also used in areas of expansive or collapsible soils to resist undesirable seasonal movements of the foundations. Under such conditions, deep foundations are designed to transfer foundation loads, including uplift or downdrag, to a level unaffected by seasonal moisture movements, Figure 3.1(k).

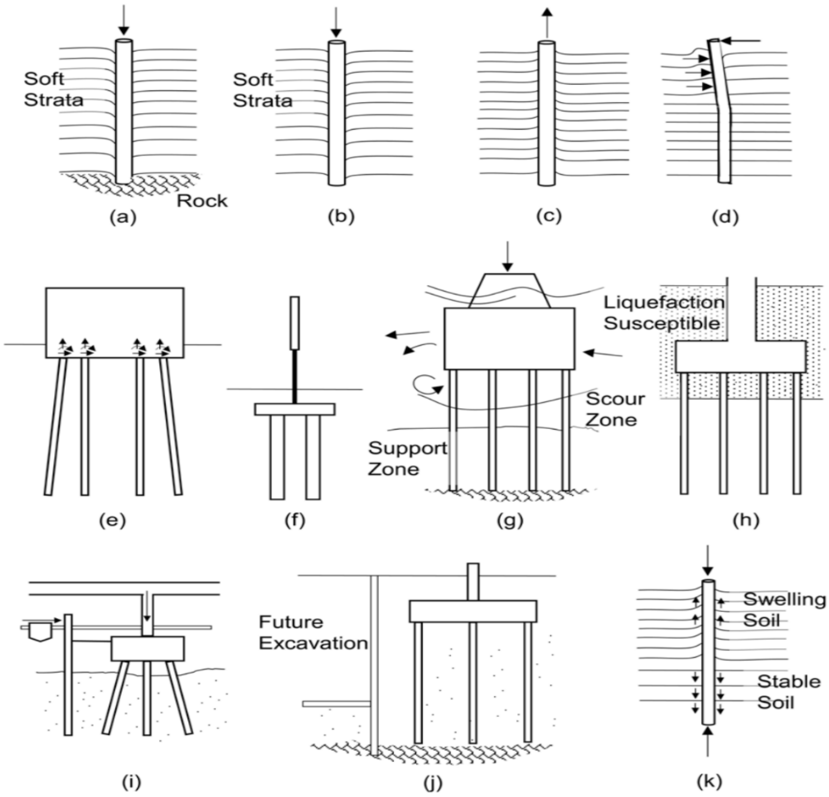


Figure 3.1 Situations of the use of Deep Foundations (Vesic 1977).

3 Typical Deep Foundations

Typical deep foundations are shown on Figure 3.2 and are listed as follows:

– Pile usually represents a slender structural element that is driven into the ground. However, a pile is often used as a generic term to represent all types of deep foundations, including a (driven) pile, (drilled) shaft, caisson, or an anchor. A pile group is used to represent various grouped deep foundations.

– Shaft is a type of foundation that is constructed with cast-in-place concrete after a hole is first drilled or excavated. A rock socket is a shaft foundation installed in rock. A shaft foundation also is called a drilled pier foundation.

– Caisson is a type of large foundation that is constructed by lowering preconstructed foundation elements through excavation of soil or rock at the bottom of the foundation. The bottom of the caisson is usually sealed with concrete after the construction is completed.

– Anchor is a type of foundation designed to take tensile loading. An anchor is a slender, smalldiameter element consisting of a reinforcement bar that is fixed in a drilled hole by grout concrete. Multistrain high-strength cables are often used as reinforcement for large-capacity anchors. An anchor for suspension bridge is, however, a foundation that sustains the pulling loads located at the ends of a bridge; the foundation can be a deadman, a massive tunnel, or a composite foundation system including normal anchors, piles, and drilled shafts.

4 Typical Bridge Foundations

Bridge foundations can be individual, grouped, or combination foundations. Individual bridge foundations usually include individual footings, large-diameter drilled shafts, caissons, rock sockets, and deadman foundations. Grouped foundations include groups of caissons, driven piles, drilled shafts, and rock sockets. Combination foundations include caisson with driven piles, caisson with drilled shafts, large-diameter pipe piles with rock socket, spread footings with anchors, deadman with piles and anchors, etc.

For small bridges, small-scale foundations such as individual footings or drilled shaft foundations, or a small group of driven piles may be sufficient. For larger bridges, large-diameter shaft foundations, grouped foundations, caissons, or combination foundations may be required. Caissons, large-diameter steel pipe pile foundations, or other types of foundations constructed by using the cofferdam method may be necessary for foundations constructed over water.

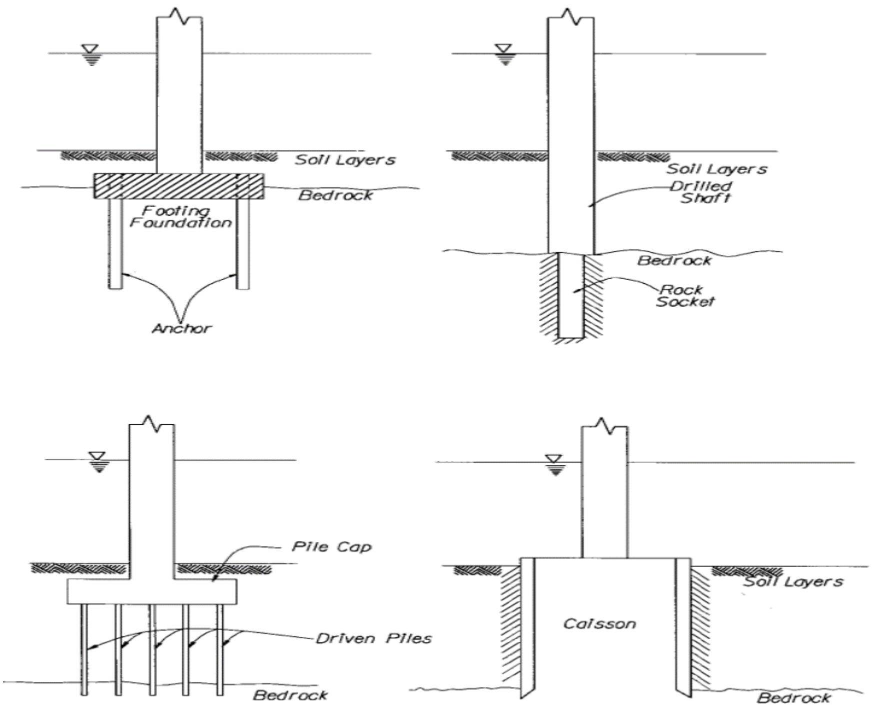


Figure 3.2 Typical Bridge Foundations.

Bridge foundations are often constructed in difficult ground conditions such as landslide areas, liquefiable soil, collapsible soil, soft and highly compressible soil, swelling soil, coral deposits, and underground caves. Special foundation types and designs may be needed under these circumstances.

5 Classification

Deep foundations have many different types and are classified according to different aspects of a foundation as listed below:

- **Geologic conditions:** Geologic materials surrounding the foundations can be soil and rock. Soil can be fine grained or coarse grained; and from soft to stiff and hard for fine-grained soil, or from loose to dense and very dense for coarse-grained soil. Rock can be sedimentary, igneous, or metamorphic; and from very soft to medium strong and hard. Soil and rock mass may possess predefined weakness and discontinuities, such as rock joints, beddings, sliding planes, and faults. Water conditions can be different, including over river, lake, bay, ocean, or land with groundwater. Ice or wave action may be of concern in some regions.
- **Installation methods:** Installation methods can be piles (driven, cast-in-place, vibrated, torqued, and jacked); shafts (excavated, drilled, and cast-in-drilled-hole); anchor (drilled); caissons (Chicago, Shored, Benoto, Open, Pneumatic, floating, closed-box, Potomac, etc.); cofferdams (sheet pile, sand or gravel island, slurry wall, deep mixing wall, etc.); or combined.
- **Structural materials :** Materials for foundations can be timber, precast concrete, cast-in-place concrete, compacted dry concrete, grouted concrete, posttension steel, H-beam steel, steel pipe, composite, etc.
- **Ground effect :**Depending on disturbance to the surrounding ground, piles can be displacement piles, low displacement, or nondisplacement piles. Driven precast concrete piles and steel pipes with end plugs are displacement piles, H-beam and umplugged steel pipes are low-displacement piles, and drilled shafts are nondisplacement piles.
- **Function:** Depending on the portion of load carried by the side, toe, or a combination of the side and toe, piles are classified as frictional, end bearing, and combination piles, respectively.
- **Embedment and relative rigidity :** Piles can be divided into long piles and short piles.

A long pile, or simply called a pile, is embedded deep enough that fixity at its bottom is established, and the pile is treated as a slender and flexible element. A short pile is relatively rigid element that the bottom of the pile moves significantly. A caisson is often a short pile because of its large cross section and stiffness. An extreme case for short piles is a spread footing foundation.

- **Cross-section :** The cross section of a pile can be square, rectangular, circular, hexagonal, octagonal, H-section; either hollow or solid. A pile cap is usually square, rectangular, circular, or bellshaped. Piles can have different cross sections at different depths, such as uniform, uniform taper, step taper, or enlarged end (either grouted or excavated).
- **Size :** Depending on the diameter of a pile, piles are classified as pin piles and anchors (100–300 mm), normal size piles and shafts (250–600 mm), large-diameter piles and shafts (600–3000 mm), caissons (600 mm and up to 3000 mm or larger), and cofferdams or other shoring construction method (very large).
- **Loading :** Loads applied to foundations are compression, tension, moment, and lateral loads. Depending on time characteristics, loads are further classified as static, cyclic, and transient loads. The magnitude and type of loading also are major factors in determining the size and type of a foundation.
- **Isolation :** Piles can be isolated at certain depth to avoid loading utility lines or other construction, or to avoid being loaded by them.
- **Inclination :** Piles can be vertical or inclined. Inclined piles are often called battered or raked piles.
- **Multiple piles :** Foundation can be an individual pile, or a pile group. Within a pile group, piles can be of uniform or different sizes and types. The connection between the piles and the pile cap can be fixed, pinned, or restrained.

Different types of foundations have their unique features and are more applicable to certain conditions than others. The advantages and disadvantages for different types of foundations are listed as follows:

5.1 Driven Precast Concrete Pile Foundations

Driven concrete pile foundations are applicable under most ground conditions. Concrete piles are usually inexpensive compared with other types of deep foundations. The procedure of pile installation is straightforward; piles can be produced in mass production either on site or in a manufacture factory; and the cost for materials is usually much less than steel piles. Proxy coating can be applied to reduce negative skin friction along the pile. Pile driving can densify loose sand and reduce liquefaction potential within a range of up to 3 diameters surrounding the pile.

However, driven concrete piles are not suitable if boulders exist below the ground surface where piles may break easily and pile penetration may be terminated prematurely. Piles in dense sand, dense gravel, or bedrock usually have limited penetration; consequently, the uplift capacity of these types of piles is very small.

Pile driving produces noise pollution and causes disturbance to the adjacent structures. Driving of concrete piles also requires large overhead space. Piles may break during driving and impose a safety hazard. Piles that break underground cannot take their design loads, and will cause damage to the structures if the broke pile is not detected and replaced. Piles could often be driven out of their designed alignment and inclination, and as a result, additional piles may be needed. Special hardened steel shoe is often required to prevent pile tips from being smashed when encountering hard rock. End bearing capacity of a pile is not reliable if the end of a pile is smashed.

Driven piles may not be a good option when subsurface conditions are unclear or vary considerably over the site. Splicing and cutting of piles are necessary when the estimated length is different from the manufactured length. Splicing is usually difficult and time consuming for concrete piles. Cutting of a pile would change the pattern of reinforcement along the pile, especially where extra reinforcement is needed at the top of a pile for lateral capacity. A pilot program is usually needed to determine the length and capacity prior to mass production and installation of production piles.

The maximum pile length is usually up to 36–38 m because of restrictions during transportation on highways. Although longer piles can be produced on site, slender and long piles may buckle easily during handling and driving. Precast concrete piles with diameters greater than 46 cm are rarely used.

5.2 Driven Steel Piles

Driven steel piles, such as steel pipe and H-beam piles are extensively used as bridge foundations, especially in the seismic retrofit projects. Having the advantage and disadvantage of driven piles as discussed earlier, driven steel piles have their uniqueness. Steel piles are usually more expensive than concrete piles. They are more ductile and flexible and can be spliced more conveniently. The required overhead is much smaller compared to driven concrete piles. Pipe piles with an open end can penetrate through layers of dense sand. If necessary, the soil inside the pipe can be taken out before further driving; small size boulders may also be crushed and taken out. H-piles with a pointed tip can usually penetrate onto soft bedrock and establish enough end bearing capacity.

5.3 Large-Diameter Driven, Vibrated, or Torqued Steel Pipe Piles

Large-diameter pipe piles are widely used as foundations for large bridges. The advantage of this type of foundation is manifold. Large-diameter pipe piles can be built over water from a barge, a trestle, or a temporary island. They can be used in almost all ground conditions and penetrate to a great depth to reach bedrock. Length of the pile can be adjusted by welding. Large-diameter pipe piles can also be used as casing to support soil above bedrock from caving in; rock sockets or rock anchors can then be constructed below the tip of the pipe. Concrete or reinforced concrete can be placed inside the pipe after it is cleaned. Another advantage is that no workers are required to work below water or ground surface. Construction is usually safer and faster than other types of foundations such as caissons or cofferdam construction.

Large-diameter pipe piles can be installed by method of driving, vibrating, or torque. Driven piles usually have higher capacity than piles installed through vibration or torque. However, driven piles are hard to control in terms of location and inclination of the piles. Moreover, once a pile is out of location or installed with unwanted inclination, no corrective measures can be applied. Piles installed with vibration or torque, on the other hand, can be controlled more easily. If a pile is out of position or inclination, the pile can even be lifted up and reinstalled.

5.4 Drilled Shaft Foundations

Drilled shaft foundations are the most versatile types of foundations. The length and size of the foundations can be tailored easily. Disturbance to the nearby structures is small compared with other types of deep foundations. Drilled shafts can be constructed very close to existing structures and can be constructed under low overhead conditions. Therefore, drilled shafts are often used in many seismic retrofit projects. However, drilled shafts may be difficult to install under certain ground conditions such as soft soil, loose sand, sand under water, and soils with boulders. Drilled shaft will generate a large volume of soil cuttings and fluid and can be mess. Disposal of the cuttings is usually a concern for sites with contaminated soils.

Drilled shaft foundations are usually comparable or more expensive than driven piles. For large bridge foundations, their cost is at the same level of caisson foundations and spread footing foundations combined with cofferdam construction. Drilled shaft foundations can be constructed very fast under normal conditions compared with caisson and cofferdam construction.

5.5 Anchors

Anchors are special foundation elements that are designed to take uplift loads. Anchors can be added if an existing foundation lacks uplift capacity, and competent layers of soil or rock are shallow and easy to reach. Anchors, however, cannot take lateral loads and may be sheared off if combined lateral capacity is not enough.

Anchors are in many cases pretensioned in order to limit the deformation to activate the anchor. The anchor system is therefore very stiff. Failure of structure resulted form anchor rupture often occurs very quickly and catastrophically. Pretension may also be lost over time because of creep in some types of rock and soil. Anchors should be tested carefully for their design capacity and creep performance.

5.6 Caissons

Caissons are large-size structures that are mainly used for construction of large bridge foundations. Caisson foundations can take large compressive and lateral loads. They are used primarily for overwater construction and sometimes used in soft or loose soil conditions, with a purpose to sink or excavate down to a depth where bedrock or firm soil can be reached. During construction, large size boulders can be removed.

Caisson construction requires special technique and experience. Caisson foundations are usually very costly, and comparable to the cost of cofferdam construction. Therefore, caissons are usually not the first option unless other types of foundations are not favored.

5.7 Cofferdam and Shoring

Cofferdam or other type of shoring system is a method of foundation construction to retain water and soil. A dry bottom deep into water or ground can be created as a working platform. Foundations of essentially any types discussed earlier can be built from the platform on top of firm soil or rock at a great depth; otherwise can only be reached by deep foundations.

Cofferdam construction is often very expensive and should only be chosen if it is favorable comparing with other foundation options in terms of cost and construction conditions.

6 Characteristics of Different Types of Foundations

The mechanisms of resistance of an individual foundation and a pile group are discussed. The function of different types of foundations is also addressed.

The complex loading on top of a foundation from the bridge structures above can be simplified into forces and moments in the longitudinal, transverse, and vertical directions, respectively (Figure 3.3). Longitudinal and transverse loads are also called the horizontal loads; longitudinal and transverse moments are called the overturning moments.

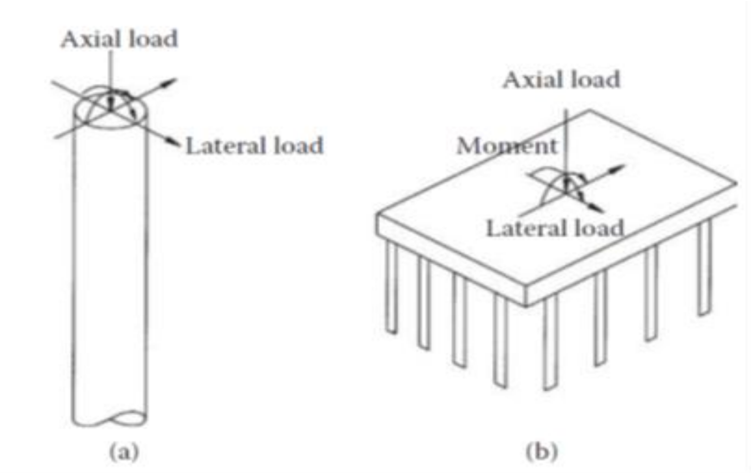


Figure 3.3 Acting Loads on Top of Piles: (a) Individual Pile, (b) Pile Group.

The resistance provided by an individual foundation is categorized in the following as seen in Figure 3.4:

End bearing: Vertical compressive resistance at the base of a foundation, distributed end bearing pressures can provide resistance to overturning moments.

Base shear: Horizontal resistance of friction and cohesion at the base of a foundation

Side resistance: Shear resistance from friction and cohesion along side of a foundation Earth pressure: mainly horizontal resistance from lateral earth pressures perpendicular to side of the foundation

Self-weight: Effective weight of the foundation

Both base shear and lateral earth pressures provide lateral resistance of a foundation, and contribution of lateral earth pressures decreases as the embedment of a pile increases. For long piles, lateral earth pressures are the main source of lateral resistance. For short piles, base shear and end bearing pressures can also contribute part of the lateral resistance.

For a pile group, through the action of the pile cap, the coupled axial compressive and uplift resistance of individual piles provides majority of the resistance to the overturning moment loading. Horizontal (or lateral) resistance can at the same time provide torsional moment resistance. A pile group is more efficient in resisting overturning and torsional moment than an individual foundation.

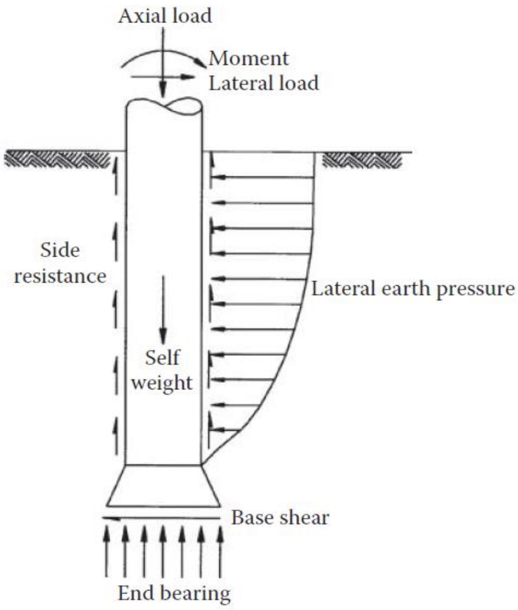


Figure 3.4 Resistance of an Individual Foundation.

7 Selection of Foundations

The two predominant factors in determining type of foundations are bridge types and ground conditions. The bridge type, including dimensions, type of bridge, and construction materials dictates the design magnitude of loads and the allowable displacements and other performance criterion for the foundations, and therefore determines the dimensions and type of its foundations. For example, a suspension bridge requires large lateral capacity for its end anchorage, which can be a huge deadman, a high capacity soil or rock anchor system, a group of driven piles, or a group of large-diameter drilled shafts. The likely foundations are deep, large-size footing using cofferdam construction, caissons, groups of large-diameter drilled shafts, or groups of large number of steel piles.

Surface and subsurface geologic and geotechnical conditions are another main factor in determining the type of bridge foundations. Subsurface conditions, especially the depths to the load-bearing soil layer or bedrock, are the most crucial factor. Seismicity over the region usually dictates the design level of seismic loads, which is often the critical and dominant loading condition. A bridge that crosses a deep valley or river certainly determines the minimum span required. Overwater bridges have limited options to choose in terms of type of foundations.

The final choice of type of foundations usually depends on cost after considering some other factors such as construction conditions, space and over head conditions, local practice, environmental conditions, schedule constraints, and so on. Certain types of foundations are excluded in the earth stage of study. For example, from the geotechnical point of view, shallow foundation is not an acceptable option if a thick layer of soft clay or liquefiable sand is near the ground surface. Deep foundations are used in cases where shallow foundations would be excessively large and costly. From constructability point of view, driven pile foundations are not suitable if boulders exist at depths above the intended firm bearing soil/rock layer.

For small bridges such as roadway overpass, for example, foundations with driven concrete or steel piles, drilled shafts, or shallow spread footing foundations may be the suitable choices. For large overwater bridge foundations, single or grouped large-diameter pipe piles, large-diameter rock socket, largediameter drilled shafts caissons, or foundations constructed with cofferdams are most likely the choice. Caissons or cofferdam construction with a large number of driven pile groups were widely used in the past. Large-diameter pipe piles or drilled shafts, in combination with rock sockets, are preferred for bridge foundations recently.

Deformation compatibility of the foundations and bridge structure is an important consideration. Different types of foundation may behave differently; therefore, same type of foundations should be used for one section of bridge structure. Diameter of the piles and inclined piles are two important factors to be considered in terms of deformation compatibility and are discussed in the following.

Small-diameter piles are more “brittle” in the sense that the ultimate settlement and lateral deflection are relatively small compared with large-diameter piles. For example, 20 small piles can have the same ultimate load capacity as two large-diameter piles. However, the small piles reach the ultimate state at a lateral deflection of 50 mm whereas the large piles, at 150 mm. The smaller piles would have failed before the larger piles are activated to a substantial degree. In other words, larger piles will be more flexible and ductile than smaller piles before reaching the ultimate state. Since ductility usually provides more seismic safety, larger diameter piles are preferred from the point of view of seismic design.

Inclined or battered piles should not be used together with vertical piles unless the inclined piles alone have enough lateral capacity. Inclined piles provide partial lateral resistance from their axial capacity; and since the stiffness in the axial direction of a pile is much larger than in the perpendicular directions, inclined piles tend to attract most of the lateral seismic loading.

Inclined piles will fail or reach their ultimate axial capacity before the vertical piles are activated to take substantial lateral loads.

8 Selection of Drilled Shafts

Drilled shafts can be installed in a variety of soil and rock profiles, and are most efficiently utilized where a strong bearing layer is present. When placed to bear within or on rock, extremely large axial resistance can be achieved in a foundation with a small footprint. The use of a single shaft support avoids the need for a pile cap with the attendant excavation and excavation support, a feature which can be important where new foundations are constructed near existing structures. Foundations over water can often be constructed through permanent casing, avoiding the need for a cofferdam. Drilled shafts can also be installed into hard, scour-resistant soil and rock formations to found below scourable soil in conditions where installation of driven piles might be impractical or impossible. Drilled shafts have enjoyed increased use for highway bridges in seismically active areas because of the flexural strength of a large diameter column of reinforced concrete. Drilled shafts may be used as foundations for other applications such as retaining walls, sound walls, signs, or high mast lighting where a simple support for overturning loads is the primary function of the foundation.

9 Bridge Foundations

For foundations supporting bridge structures, conditions favorable to the use of drilled shafts include the following:

- Cohesive soils, especially with deep groundwater. For these type soil conditions, drilled shafts are easily constructed and can be very cost effective (Figure 3.5).
- Stratigraphy where a firm bearing stratum is present within (30 m) of the surface. Drilled shafts can provide large axial and lateral resistance when founded on or socketed into rock or other strong bearing strata.

Figure 3.5 Construction of Drilled Shaft in Dry, Cohesive Soils

- Construction of new foundations where a small footprint is desirable. For a widening project or an interchange with "flyover" ramps or other congested spaces, a single drilled shaft under a single column can avoid the large footprint that would be necessary with a group of piles. A single shaft can also avoid the cost of shoring and possibly dewatering that might be required for temporary excavations. Construction of drilled shafts can often be performed with minimal impact on nearby structures. Figure 3.6 illustrates some examples of these types of applications.

Figure 3.6 Drilled Shafts for Bridge Foundations where Small Footprint is desirable.

- Construction of foundations over water where drilled shafts may be used to avoid construction of a cofferdam. Figure 3.7 illustrates a two column pier under construction in a river with a single shaft supporting each column.

Figure 3.7 Drilled Shafts for Individual Column Support over Water

- Foundations with very high axial or lateral loads. Figure 3.8 shows construction of a 5-shaft group with a waterline footing for a bridge with large foundation loads in relatively deep water.

Figure 3.8 Group of Drilled Shafts for Large Loads

- Foundations with deep scour conditions where driven piles may be difficult to install. Figure 3.9 is from a bridge in Arizona. The original piles had been driven to refusal but subsequently one of the foundations had been lost due to scour.

Figure 3.9 Drilled Shafts Installed for Deep Scour Problem

- Construction of new foundations with restricted access or low overhead conditions. Often, high capacity drilled shaft foundations can be constructed in these circumstances with specialty equipment. Construction of new foundations for a replacement structure in advance of demolition of existing structures can be used to reduce the impact of construction on the traveling public. Figure 3.10 shows drilled shafts with low headroom.

Figure 3.10 Drilled Shafts with Low Headroom

10 Axial, Lateral, and Moment Capacity

Deep foundations can provide lateral resistance to overturning moment and lateral loads, and axial resistance to axial loads. Part or most of the moment capacity of a pile group are provided by the axial capacity of individual piles through pile cap action. The moment capacity depends on the axial capacity of the individual piles, geometry arrangement of the piles, rigidity of the pile cap, and rigidity of connection between the piles and the pile cap. Design and analysis is often concentrated on the axial and lateral capacity of individual piles.

11 Other Design Issues

Proper foundation design should consider many factors regarding the environmental conditions, type of loading conditions, soil and rock conditions, construction, and engineering analyses, including:

- Various loading and loading combinations, including the impact loads of ships or vehicles
- Earthquake shaking
- Liquefaction
- Rupture of active fault and shear zone
- Landslide or ground instability
- Difficult ground conditions such as underlying weak and compressible soils
- Debris flow
- Scour and erosion
- Chemical corrosion of foundation materials
- Weathering and strength reduction of foundation materials
- Freezing
- Site contamination condition of hazardous materials

- Influence upon and by nearby structures

12 Summary of Design Methods for Deep Foundations

Table 1 Type 1: Driven Pile

Design for	Soil	Method and Author
End Bearing	Clay	Nc method (Skempton, 1951)
		Nc method (Goudreault and Fellenius, 1994)
		CPT methods (Meyerhof,1956; Davies et al.,1988;Schmertmann, 1978)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
End Bearing	Sand	Nq method with critical depth concept (Meyerhof, 1976)
		Nq method (Berezantzev et al., 1961)
		Nq method (Goudreault and Fellenius, 1994)
		Nq by others (Janbu, 1976; Terzaghi, 1943; Vesic, 1967)
		Limiting Nq values (API, 2000; de Ruiter and Beringen, 1978)
		Value of φ (Kishida, 1967; Kulhawy, 1983; Mitchell and Lunne,1978)
		SPT (Meyerhof, 1956, 1976)
		CPT methods (Meyerhof,1956;Davies et al.,1988; Schmertmann, 1978)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
End Bearing	Rock	(CGS, 1992)
Side Resistance	Clay	α-method (Tomlinson, 1957, 1971)
		α-method (API, 2000)
		β-method (Goudreault and Fellenius, 1994)
		λ-method (Kraft et al., 1981; Vijayvergiya and Focht, 1972)
		CPT methods (Meyerhof, 1956; Davies et al., 1988; Schmertmann, 1978)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
		SPT (Dennis, 1982)
Side Resistance	Sand	α-method (Tomlinson, 1957, 1971)
		β-method (Burland, 1973)
		β-method (Goudreault and Fellenius, 1994)
		CPT method (Meyerhof, 1956; Davies et al., 1988; Schmertmann, 1978)

		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
		SPT (Meyerhof, 1956, 1976)
Side And End	All	Load test: ASTM D 1143, static axial compressive test
		Load test: ASTM D 3689, static axial tensile test
		Sanders' pile driving formula (1850; Poulos and Davis, 1980)
		Danish pile driving formula (Sorensen and Hansen, 1957)
		Engineering News formula
		Dynamic formula—WEAP Analysis
		Strike and restrike dynamic analysis
		Interlayer influence (Meyerhof, 1976)
		No critical depth (Fellenius, 1994; Kulhawy, 1984)
Load-Settlement	Sand	(Vesic,1970)
		(Mosher,1984; Vijayvergiya,1977)
Load-Settlement	All	Theory of elasticity, Mindlin's solutions (Poulos and Davis, 1980)
		Finite element method (Desai and Christian, 1977)
		Load test: ASTM D 1143, static axial compressive test
		Load test: ASTM D 3689, static axial tensile test

Table 2 Type 2: Drilled Shaft

Design for	Soil	Method and Author
End Bearing	Clay	Nc method (Skempton, 1951)
		Large base (O'Neil and Sheikh, 1985; Reese and O'Neil, 1988)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
End Bearing	Sand	(Touma and Reese, 1972)
		(Meyerhof, 1976)
		(Reese and Wright, 1977)
		(Reese and O'Neil, 1988)
		SPT (Meyerhof, 1956, 1976)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
End Bearing	Rock	(CGS, 1992)
		Pressure meter (CGS, 1992)
Side Resistance	Clay	α-method (Reese and O'Neil, 1988)
		α-method (Skempton, 1959)

		α-method (Weltman and Healy, 1978)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
Side Resistance	Sand	(Touma and Reese, 1972)
		(Meyerhof, 1976)
		(Reese and Wright, 1977)
		β-method (O'Neil and Reese, 1978, Reese and O'Neil, 1988)
		SPT (Reese and O'Neil, 1988)
		CPT (Bustamante and Gianeselli, 1982; CGS, 1992)
Side Resistance	Rock	Coulombic (McVay et al., 1992)
		Coulombic (Townsend, 1993)
		SPT (Crapps, 1986)
		(Gupton and Logan, 1984)
		(Reynolds and Kaderabek, 1980)
		(Carter and Kulhawy, 1988; Kulhawy and Phoon, 1993)
		(Horvath and Kenney, 1979)
Side and End	Rock	(O'Neil et al., 1996)
		(Williams et al., 1980)
		(Rosenberg and Journeaux, 1976)
		(Pells and Turner, 1979, 1980)
		(Rowe and Armitage, 1987a, 1987b)
		FHWA (Reese and O'Neil, 1988)
Side and End	All	Load test (Osterberg, 1989)
Load-Settlement	Sand	(Reese and O'Neil, 1988)
	Clay	(Reese and O'Neil, 1988)
		Clay (Woodward et al., 1972)
	All	Load test (Osterberg, 1989)

Table 3 Type 3: All Types

Design for	Soil	Method and Author
Lateral Resistance	Clay	Broms' method (Broms, 1964a)
Lateral Resistance	Sand	Broms' method (Broms, 1964b)
Lateral Resistance	All	*p-y* method (Reese, 1983)
Lateral Resistance	Clay	*p-y* response (Matlock, 1970)
	Clay(w/water)	*p-y* response (Reese et al., 1975)

	Clay(w/o water)	*p-y* response (Welch and Reese, 1972)
Lateral Resistance	Sand	*p-y* response (Reese et al., 1974)
Lateral Resistance	All	*p-y* response (American Petroleum Institute [API], 2000)
		p-y response for inclined piles (Awoshika and Reese, 1971; Kubo,1965)
		p-y response in layered soil
		p-y response (NAVFAC DM7.02, 1986)
Lateral Resistance	Rock	*p-y* response (O'Neil et al., 1996)
Load-Settlement	All	Theory of elasticity method (Poulos and Davis, 1980)
		Finite difference method (Seed and Reese, 1957)
		General finite element method (FEM)
		FEM dynamic
End Bearing	All	Pressuremeter method (Menard, 1975; Vesic, 1972)
Lateral Resistance	All	Pressuremeter method (Menard, 1975)
		Load test: ASTM D 3966

Table 4 Type 4: Group Type

Design for	Soil	Method and Author
Theory	All	Elasticity approach (Poulos and Davis, 1980)
		Elasticity approach (Focht and Koch, 1973)
		Two dimensional group (Reese and Matlock, 1966)
		Three dimensional group (Reese and O'Neil, 1967)
Lateral g-Factor	All	(CGS, 1992)
		(Dunnavavnt and O'Neil, 1986)

Chapter 4

Soil-Structure Interaction

1. Introduction

While there are several programs that can perform substructure analysis, such as LPile (Ensoft 2018) and GROUP (Ensoft 2018), FB-MultiPier (BSI 2019) is capable of doing both substructure and direct analysis. This was an important factor in deciding which program to use.

1.1. Overview of FB-MultiPier

FB-MultiPier is a hybrid finite element analysis program developed by the Bridge Software Institute (BSI), shown in Figure 4.1, which is headquartered at the University of Florida in Gainesville, FL,USA. It is capable of modeling multiple bridge pier structures that are interconnected by single representative bridge spans. The full structure can be subjected to a full array of AASHTO load types in a static analysis or time varying load functions in a dynamic analysis (BSI 2019).

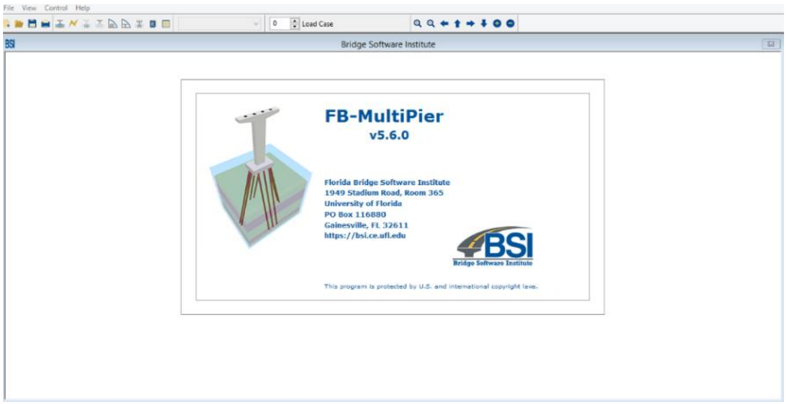


Figure 4.1 FB-MultiPier Program (BSI 2019)

The structural elements that it is capable of simulating include foundation element(s) (piles and drilled shafts), pile cap(s), column, and pier cap. All of the structural elements can be uniquely modeled by the user. The program also provides standard sections for many common foundation elements (H-pile, drilled shaft, prestressed concrete pile, pipe pile, etc.). For the soil-foundation interaction, FB-MultiPier uses axial (t-z, Q-z), lateral (p-y), and torsional (T□□□ nonlinear spring functions (soil springs). FB-MultiPier employs several soil spring functions to characterize the soil stiffness as well as the capability to enter a customized set of ten curve points if none of the default soil springs are suitable.

FB-MultiPier uses an iterative solution method to solve for the structural displacements. This method follows a secant approach where FB-MultiPier finds the stiffness of the soil and structure for a computed set of displacements, assembles a stiffness matrix, and then solves for a new set of displacements. Convergence is achieved when the system is in static equilibrium.

When first opening FB-MultiPier, the user must open an existing file or select a new problem type. If a new problem type is selected, a default file is automatically loaded and displayed (BSI 2019). Figure 4.2 shows the home screen of FB-MultiPier. The top left window is the Model Data window where most of the information is entered (BSI 2019). The top right window is the Pile Edit window and it shows the pile group in plan. The bottom left window is the Soil Edit window where the soil stratigraphy is shown. A 3-D view of the pier structure is shown in the bottom right pane. This graphical user interface allows the user to see the development of the model as it is being built, which can help find mistakes and accelerate the process (BSI 2019).

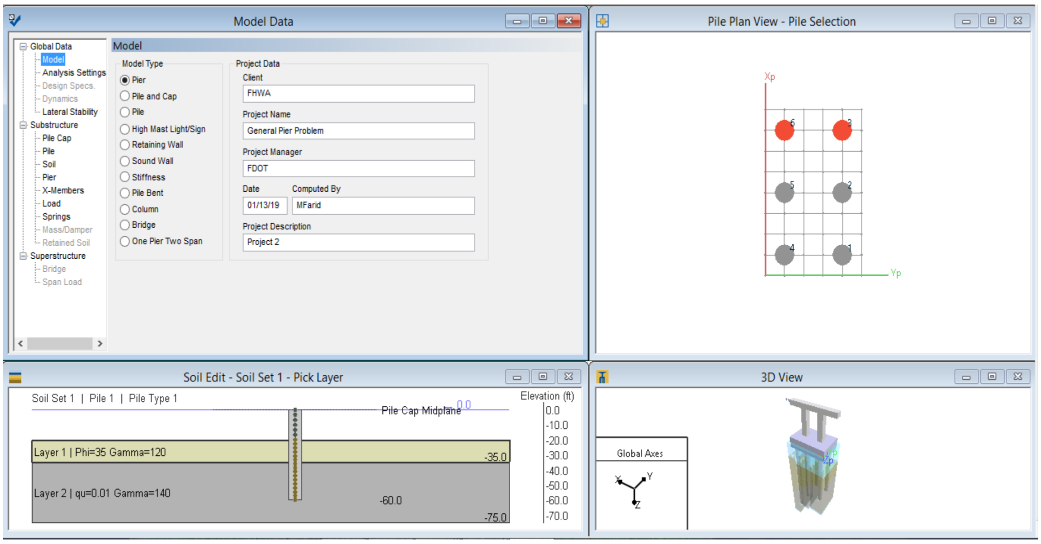


Figure 4.2 FB-MultiPier Editor Window

The following sections provide a brief introduction to the system processes and various models employed by FB-MultiPier

1.2. Soil Modeling

FB-MultiPier is capable of modeling multiple soil sets and layers within a model. This is important as site conditions can vary within a few feet. There are several important soil properties that are required as input parameters within the program such as: Young's modulus, Poisson's ratio, shear modulus, angle of internal friction, undrained strength, subgrade modulus, and the water table elevation (BSI 2019). However, depending on what soil model is selected, other properties, such as shear strain, unit skin friction and ultimate tip resistance may be required.

1.3. Structural Modeling

FB-MultiPier is capable of modeling complex structural components. The structural components are modeled by inputting the pier geometry (pier height, pier cap cantilever length, column spacing and offset, number of piers, and pier cap slope), cross-section parameters, and taper data (if applicable). The program has default cross sections or the user can model a custom one. The full cross section option requires reinforcement details and material properties. The sectional properties are calculated internally.

The program can conduct linear or non-linear analysis for both the pile and pier (column and cap). If linear behavior is selected, it is assumed the behavior is purely linear elastic and deflections do not cause secondary moments. If non-linear analysis is selected, the program accounts for second order effects (p-delta) as well as stiffness changes within the structure, such as cracking of concrete, and it uses either user defined or default stress-strain curves. P-delta effects occur when the axial force becomes eccentric within the element due to displacements of one end of the element relative to the other, causing an out-of-balance moment within the member. The default non-linear stress strain curve of concrete is a function of compressive strength and the Young's modulus of elasticity of the concrete. The default stress-strain curve for mild steel, such as an H-pile, is elastic-perfectly plastic and a function of Young's modulus of elasticity and the yield strength. These default stress-strain curves are shown in Figures 4.3 and 4.4. Concrete modulus is also needed for developing the concrete models in FB-MultiPier.

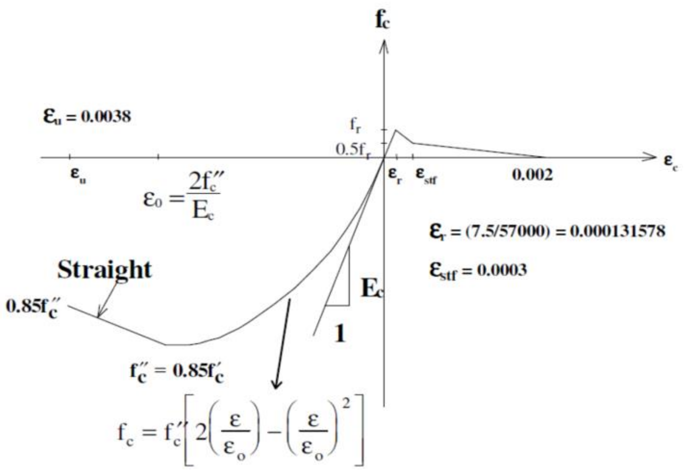


Figure 4.3 Hognestad model for concrete (Hognestad et al. 1955)

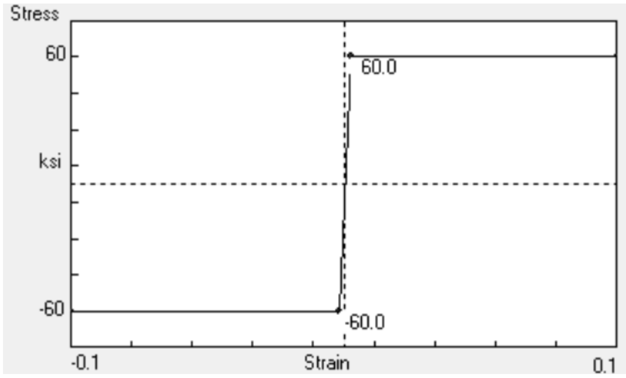


Figure 4.4 Default stress-strain curve for 60 ksi steel (BSI 2019)

1.4. Foundation Modeling

The foundation elements are modeled like that of the structure elements as previously mentioned. There are default options as well as a user defined option available. The option to model multiple pile sets is also available. For example, if the design calls for one drilled shaft tip elevation at 250 feet and the other at 265 feet, FB-MultiPier can specify 2 (or more) pile sets. This is an important option, as tip elevations or pile types can be different for a large pier structure.

1.5. Pile Cap Modeling

The pile cap is modeled based on the concrete’s Young’s modulus of elasticity, Poisson’s ratio, thickness, and unit weight of the pile cap material (usually concrete). To avoid stress concentrations at the base column node where it connects to the pile, FB-MultiPier spreads the load to the four adjacent nodes on the pile cap using rigid connectors built in to the program (BSI 2019). The user has the option to choose whether to treat the pile-to-cap connection as pinned or rigid (referred within the program as fixed).

The pile cap can also be a factor in lateral and axial capacity within the program. A simple parametric study was done to compare a pile cap just above the ground surface and a fully embedded pile cap. It was found that the lateral deflections decreased significantly when the pile cap was embedded. Though this is generally correct, the soil resistance around the pile cap may change during construction depending on the techniques used and depth of embedment. Therefore, it is up to the engineer to determine the “as built” strength of the soil surrounding the pile cap. In the analysis edit window, the option to include axial bearing effects of the pile cap is available. is to the discretion of the engineer whether to use it or not. Typically, pile foundations are designed with the assumption that the axial forces would be resisted by the piles alone.

2. Soil Properties

Following are the important soil properties required as input parameters.

1. Young's Modulus
2. Poisson's Ratio
3. Shear Modulus
4. Angle of Internal Friction
5. Undrained Strength
6. Subgrade Modulus

2.1. Young's Modulus

The young's modulus, of soils, can be obtained from following empirical equations:

For Sand

$$E = \alpha * P_a * N_{60} \quad \textbf{\textit{(psf)}} \tag{4.1.}$$

where

α = 5 for sands with fines

10 for clean normally consolidated sand

15 for clean overconsolidated sand

P_a = Atmospheric pressure (≈ 2000 psf)

N_{60}= Corrected SPT blow-count (blows/ft)

$$E = k * B * (1 - \nu^2) \quad \textbf{\textit{(psf)}} \tag{4.2.}$$

where

k = Subgrade modulus (pcf)

B = Width of pile (ft)

ν = Poisson's ratio

$$E = k * z \quad \textbf{\textit{(psf)}} \tag{4.3.}$$

where

k = Subgrade modulus (pcf)

z = Depth below ground surface (ft)

For Clay

$$E = \beta * C_u \quad \textbf{\textit{(psf)}} \qquad (4.4)$$

where

β = range of beta is shown in the table below

C_u= undrained shear strength (psf)

Table 5 Range of β for Clay

Plasticity Index	β				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
< 30	1500 – 600	1380 – 500	1200 – 580	950 – 380	730 – 300
30 to 50	600 – 300	550 – 270	580 – 220	380 – 180	300 – 150
> 50	300 – 150	270 – 120	220 – 100	180 – 90	150 – 75

2.2. Poisson's Ratio

The following typical values may be used for the Poisson's ratio ν for soils:

ν = 0.2 to 0.45 (Sand)

= 0.4 to 0.5 (Clay)

or a spatial average, for the values of ν over depth may be used for soils consisting of both sand and clay.

2.3. Shear Modulus

The shear modulus of soils, *G*, is a function of soil type, past loading, and geological history. It is recommended that *G* be obtained from insitu tests such as dilatometer, CPT, and/or SPT. Note that the equations presented below constitute relatively broad descriptions of estimating soil shear modulus, drawing upon theory of elasticity and empirical methods. Engineering judgment should be used in deciding on applicability of the specific formulations listed below, or one of many available alternative formulations. For example, for relatively undisturbed soils, Table 6-6 of Kramer (1996) may be of use in estimating representative values of shear modulus.

G can be computed from Young's Modulus , E and Poisson's ratio , v, from the following correlation:

$$G = \frac{E}{2(1+v)} \tag{4.5}$$

For Sand

$$G_{max} = 35 * (N_{60})^{0.68} \text{ (ksf)} \tag{4.6}$$

where

G_{max} = maximum shear modulus (ksf)

N_{60} = corrected SPT blow-count (blows/ft)

When applicable, use values of Young's Modulus, E from Eqn 4.1 to Eqn 4.4 to calculate shear modulus for sand. Note that alternative formulations may be more applicable, depending on soil/site conditions (e.g., empirical formulations listed in Table 6-6 of Kramer, 1996).

$$G = \frac{E}{2(1+v)} = \frac{\alpha * P_a * N_{60}}{2(1+v)} = \frac{k * B * (1-v^2)}{2(1+v)} = \frac{k * z}{2(1+v)} \text{ (psf)} \tag{4.5}$$

where

α = 5 for sand fines

10 for clean normally consolidated sand

15 for clean overconsolidated sand

P_a= atmospheric pressure(≈ 2000 psf)

N_{60} = corrected SPT blow-count (blows/ft)

k = subgrade modulus (pcf)

B = width of pile (ft)

v = poisson's ratio

z = depth below ground surface (ft)

For Clay

When applicable, use values of Young's Modulus, E from Eqn: 4.4 in Eqn 4.5 to calculate shear modulus for clay. Alternative formulations, such as the empirical relationships listed in Table 6-6 of Kramer, 1996 may also be applicable.

$$G = \frac{E}{2(1+\nu)} = \frac{\beta * C_u}{2(1+\nu)} \text{ (psf)} \tag{4.6}$$

where

β = range of beta shown in Eqn: 12.5.D

C_u= undrained shear strength (psf)

2.4. Angle of Internal Friction

Angle of internal friction, φ', can be computed from SPT N values using the following empirical correlation:

Table 6 Correlation between SPT N values and Angle of internal friction, φ',

N'		4	10	30	50
φ'	25-30	27-32	30-35	35-40	38-43

$$N' = C_N N \tag{4.7}$$

where

C_N = Correction for overburden pressure

FHWA 96 uses the correction by Peck, et al. (1974):

$$C_N = 0.77\ log_{10}\left(\frac{20}{\sigma'_v\,(tsf)}\right) = 0.77\ log_{10}\left(20\,\frac{1915.2}{\sigma'_v\,(kPa)}\right) \tag{4.8}$$

valid only for $\sigma'_v \geq 0.25$ tsf (24 kPa) (Bowles, 1977)

Normalizing for atmospheric pressure (pa): (1 atm = 101.3 kPa = 1.06 tsf)

$$C_N = 0.77 log_{10}\left(20\,\frac{pa}{\sigma'_v}\right) \tag{4.9}$$

Larger values should be used for granular material with 5% or less of fine sand and silt.

For numerical implementation, the average correlation can be expressed as

$$\varphi' = a * N' + b \tag{4.10}$$

Where:

Table 7 Constants a and b to determine Angle of internal friction, φ',

N'	a	b
0 - 10	0.50	27.5
10 - 30	0.25	30.0
30 - 50	0.15	33.0
50 -	0	40.5

2.5. Undrained Strength

Estimates of undrained shear strength, c_u can be made using the correlations of q_u with SPT N-values (see the Figure below).

$$c_u = \frac{q_u}{2} \tag{4.11}$$

where

q_u = Unconfined Compressive Strength

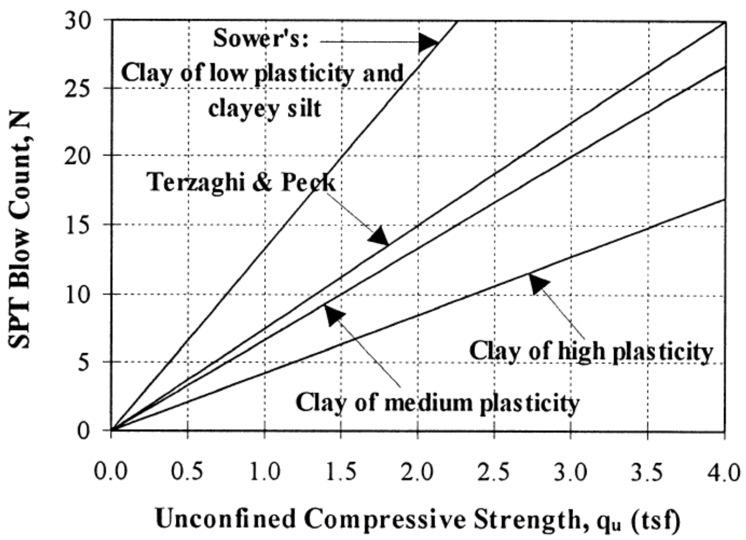


Figure 4.5 Correlations between SPT N-value and Unconfined Compressive Strength

2.6. Subgrade Modulus

Subgrade modulus, k (F/L^3) of cohesionless soil can be estimated from empirical correlations.

Correlations for submerged cohesionless soils located in Florida

For submerged cohesionless soils, use SPT-N values to find k (Figure 4.2);[FDOT SFH (Appendix B) 2017].

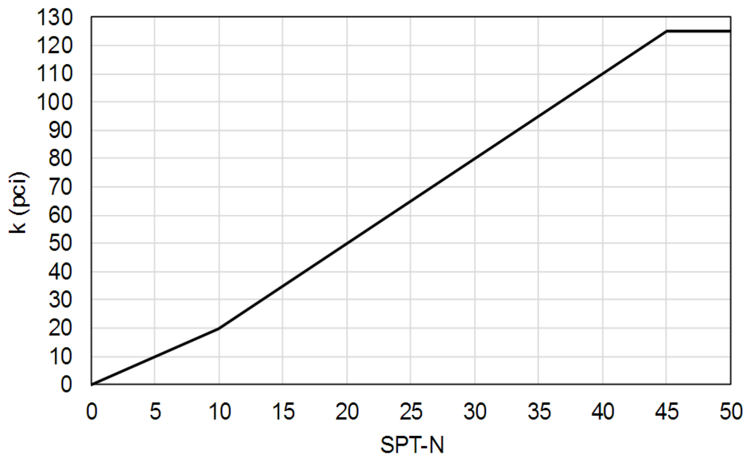


Figure 4.6 SPT-N versus k (pci) for Submerged Cohesionless Soil (FDOT 2017)

Correlations for cohesionless soils (General)

If Figure 4.6 is not applicable, use relative density to find *k*, as listed in the Tables 4.4 and 4.5 below (FHWA COM624P 2.0 Manual, 1993).

Table 8 **Representative values of k for submerged sand (FHWA 1993)**

Relative density	Loose	Medium	Dense
Recommended k (pci)	20	60	125
Recommended k (kN/m^3)	5429	16287	33931

Table 9 **Representative values of k for sand above water table (FHWA 1993)**

Relative density	Loose	Medium	Dense
Recommended k (pci)	25	90	225
Recommended k (kN/m^3)	6786	24430	61076

3. Soil Pile Interaction

This section defines input parameters and soil models available for lateral, axial, torsional, rotational, and tip resistance. Lateral soil-structure interaction is modeled with nonlinear p-y curves. Axial interactions are modeled with hyperbolic τ-z curves. Coupling can be included between axial and lateral resistance. Tip resistance is modeled with compression-only non-linear q-z curves as presented in the Axial Soil Resistance section. All soil-structure interaction calculations pertain to immediate settlement. Following are the categories of soil pile interaction

1. Group Interaction
2. Lateral Soil Resistance
3. Axial Soil Resistance
4. Torsional Soil Resistance

3.1. Group Interaction

When a group of piles is subject to vertical or lateral loads (i.e. wind, earthquake, etc.) the group vertical or lateral resistance is generally not equal to the sum of the individual pile resistance. Generally, the group resistance is less than the individual pile resistance and is a function of pile location within the group, and pile spacing.

The approach recommended by Hannigan et. al. (2006) with P-Multipliers listed in the AASHTO LRFD specification has been implemented in the program. Separate listings of p-multipliers can then be specified for lead/trail rows in the Xp direction and lead/trail rows in the Yp direction. The program identifies the lead/trail rows in each direction (with unique collections of p-multipliers assigned to Xp springs and Yp springs), as part of the equilibrium iterations, based on the computed motions of the pile head nodes under the applied loading.

The following P-Multipliers are recommended for lateral loading at 3D pile spacing: 0.8, 0.4, 0.3, 0.3, …..0.3 where 0.8 is the lead row and 0.3 is the trail row value.

For 5D pile spacing the following P-Multipliers are recommended: 1.0, 0.85, 0.7, 0.7, …, 0.7 where 1.0 is the lead row and 0.7 is the trail row value.

Note: The program will apply the P-Multipliers to the correct pile rows (lead to trail)based on the direction the piles move. The P-Multipliers are always given in trail to lead order. In this way, P-Multipliers can be input independent of the applied loading (i.e., the program automatically determines the lead and trail rows as part of the analysis).

3.2. Lateral Soil Resistance

The following p-y models are available for modeling lateral soil resistance:

3.2.1. Sand (O'Neill)

Murchison and O'Neill (1984) recommended a hyperbolic p-y relationship for sand (Figure 4.7) for both short-term static and cyclic loading conditions:

$$p = \eta A p_u tanh\left[\left(\frac{kz}{\eta A p_u}\right) y\right] \quad (4.12)$$

where

p = Horizontal sand resistance per unit length

y = Horizontal displacement

η = Pile shape factor, which is equal to 1 for circular piles

A = Factor depending on the loading type

p_u = Horizontal ultimate sand resistance per unit length

k = Initial coefficient of subgrade reaction [F/L^3]; refer to Para 1.6 Subgrade Modulus for typical values for sand above or below water table

z = Depth

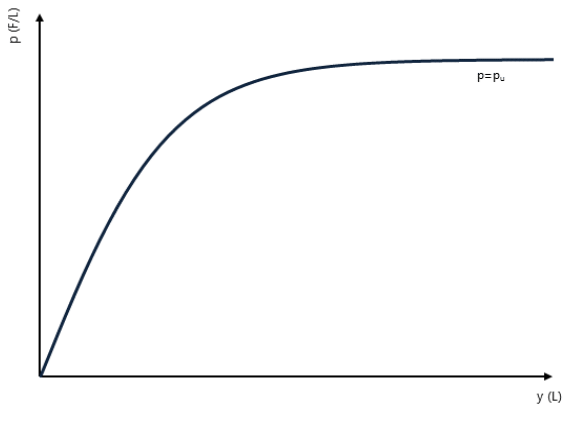


Figure 4.7 P-y curve for Sand (O'Neill and Murchinson, 1983)

3.2.2. Sand (Reese)

Based on the results obtained from an extensive load testing program on pipe piles in Texas, Reese et al., (1974) developed a p-y relationship for short-term static and cyclic loading of sands. To construct this p-y relationship, p and y values should be obtained at three distinct points k, m and u (Figure 4.8). The p-y relationship between these points is linear except the section between the points k and m, where the relationship is parabolic.

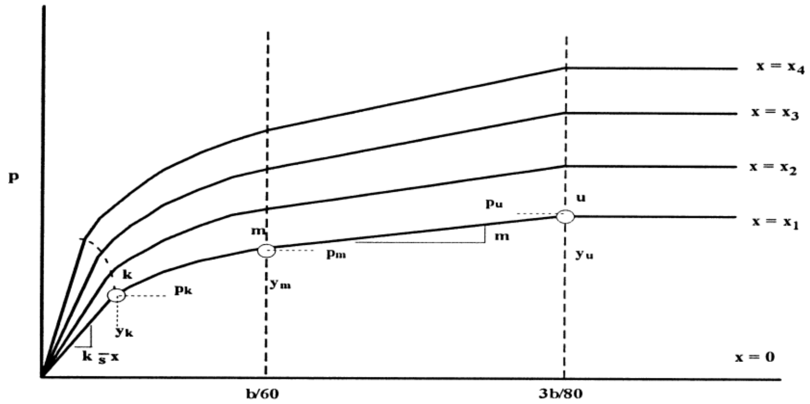


Figure 4.8 P-y Curve for Static and Cyclic Loading of Sand (After Reese et al, 1974)

3.2.3. Clay (O'Neill)

O’Neill and Gazioglu (1984), O’Neill and Dunnavant (1984), and Dunnavant and O’Neill (1985) recommended a p-y relationship for clay for both short-term static and cyclic loading conditions. Shown in the figures 4.9 and 4.10 are both the short-term static and cyclic p-y relationships. The engineer must supply the undrained strength, c, the strains at 50% failure (ε_{50}) and 100% of failure (ε_{100}) from an unconfined compression test on clay samples.

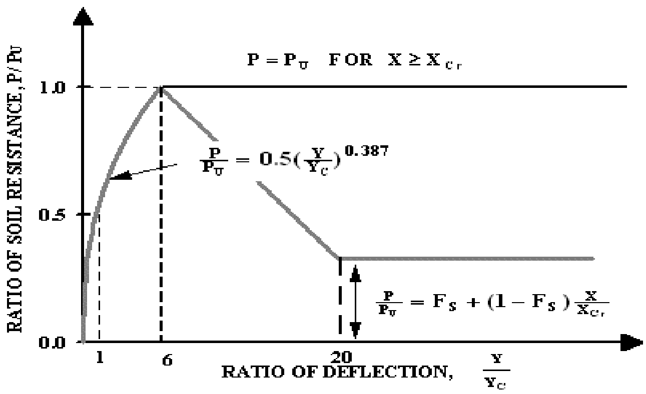


Figure 4.9 P-y Curve for Clay Short-term Static Loading Condition (O’Neill and Dunnavant, 1984)

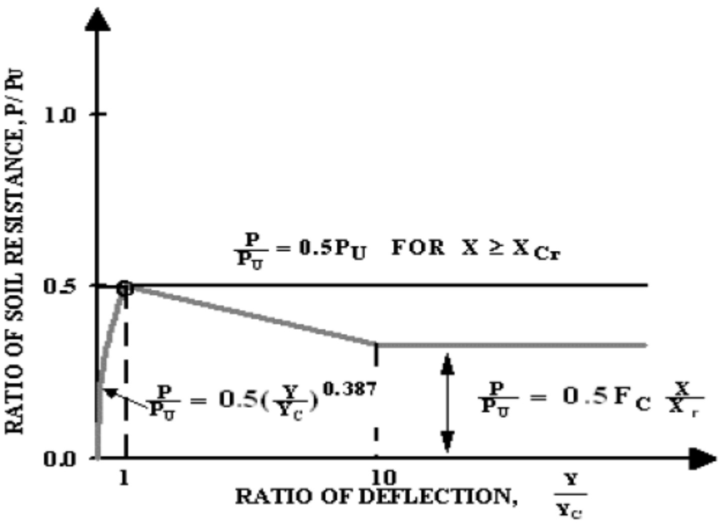


Figure 4.10 P-y Curve for Clay for Cyclic Loading Condition (O'Neill and Dunnavant, 1984)

3.2.4. Clay (Soft, Matlock)

Matlock (1970) performed lateral load tests represented by p-y curves of soft clay below the water table. The p-y curves for both static and cyclic response are shown in Figures 4.11 and Figure 4.12 respectively. Two types of short-term static and cyclic lateral load tests were conducted.

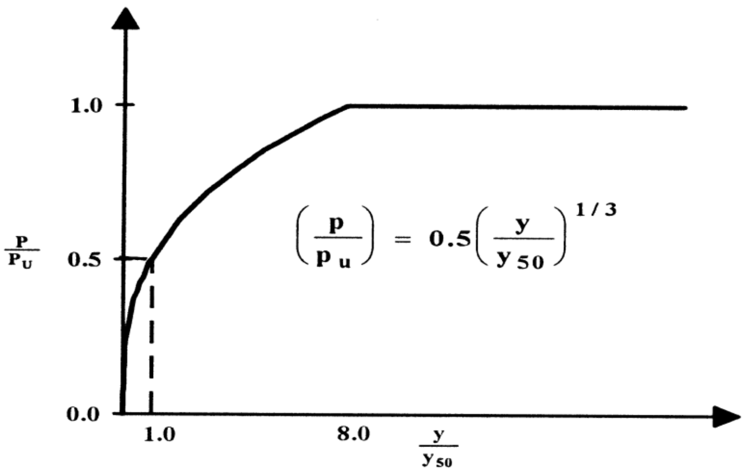


Figure 4.11 P-y Curve for Soft Clay below Water Surface (Static Loading)

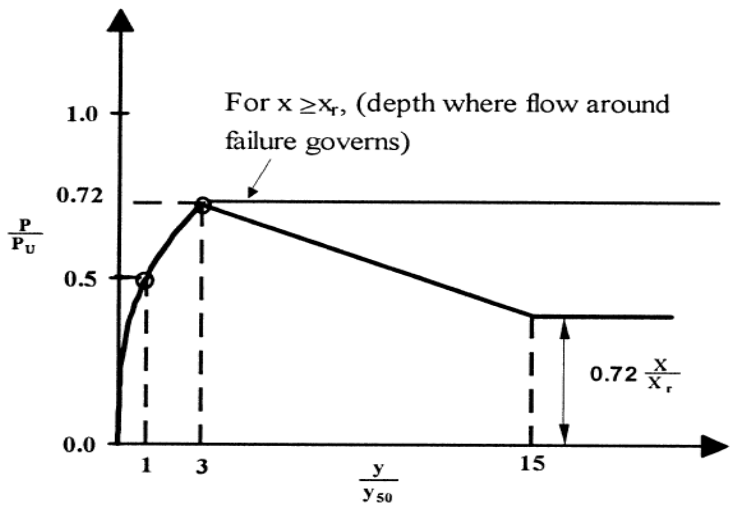


Figure 4.12 P-y Curve for Soft Clay below Water Surface (Cyclic Loading)

3.2.4.1. Short-term static loading condition

Matlock (1970) recommended the following p-y relationship for short-term static loading condition:

$$p = 0.5p_u \left(\frac{y}{y_{50}}\right)^{1/3} \tag{4.13}$$

where

p = horizontal soft clay resistance per unit length

p_u = horizontal ultimate soft clay resistance per unit length of pile

y = horizontal displacement in mm

y_{50} = displacement at one-half of the ultimate soft clay resistance

3.2.4.2. Cyclic loading condition

For cyclic loading conditions, the maximum horizontal soft clay resistance per unit length (p) is limited to $0.72p_u$.

3.2.5. Reese's Stiff Clay below Water Table

Welch and Reese (1972); Reese and Welch (1975) performed lateral load tests in Texas on a fully instrumented drilled shaft, 915 mm (36 in) in diameter, and driven into stiff clay with undrained shear strength of approximately 105 kPa (2200 psf) in the upper 6 m (19.7 ft) of the site. Two types of short-term static and cyclic lateral load tests were conducted.

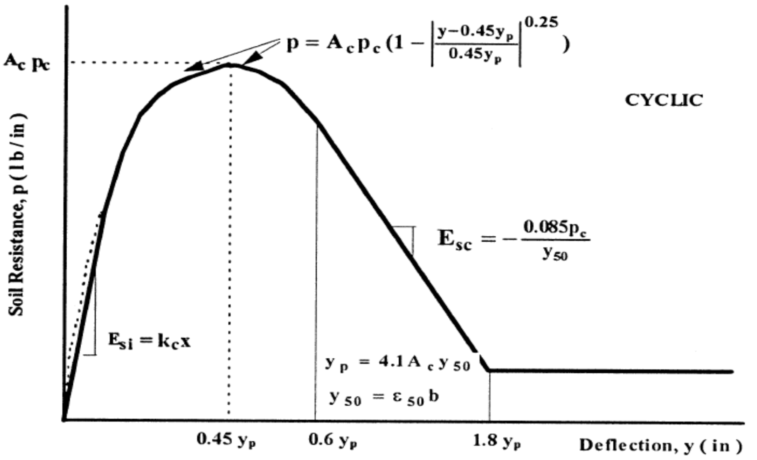


Figure 4.13 Reese et al (1975) Cyclic p-y Curve for Stiff Clay below Water Table

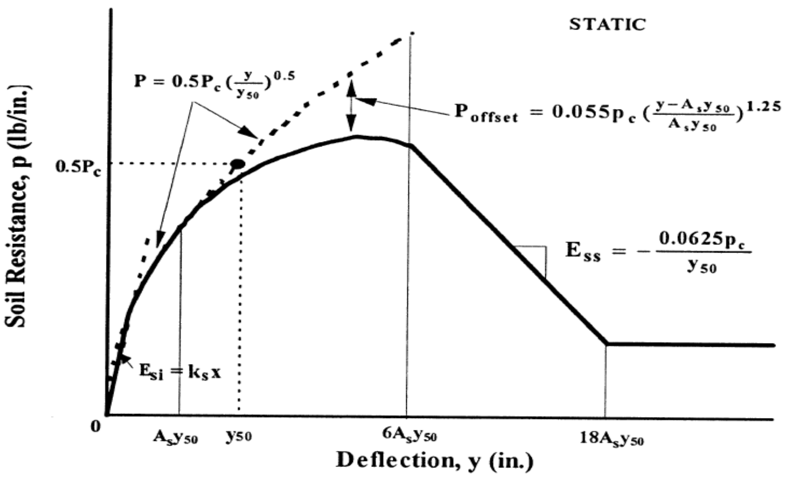


Figure 4.14 Reese et al (1975) Static p-y Curve for Stiff Clay below Water

3.2.6. Reese and Welch's Stiff Clay Above Water Table

Reese and Welch 1975 performed p-y model for stiff clay above the water table. The p-y curves for both static and cyclic response are shown in Figures 4.11 and Figure 4.12 respectively. Two types of short-term static and cyclic lateral load tests were conducted.

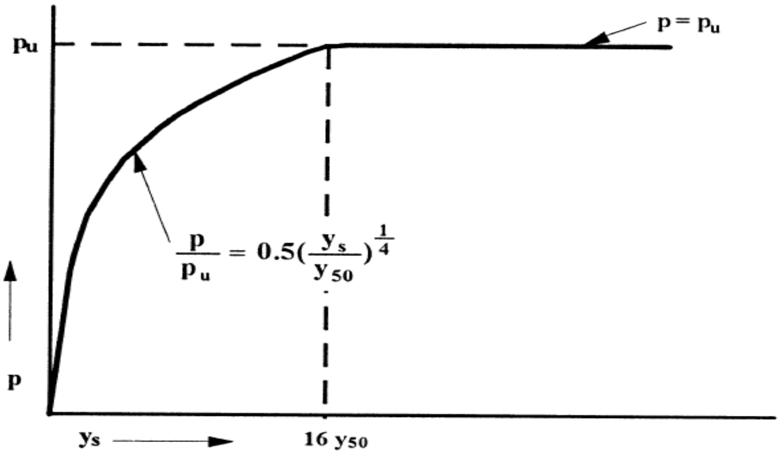


Figure 4.15 Reese and Welch (1972) Static p-y Curve for Stiff Clay Above Water Table

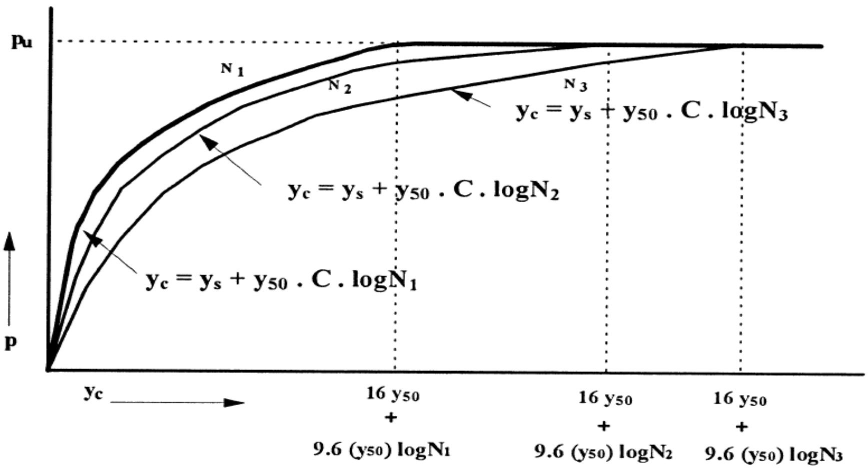


Figure 4.16 Reese and Welch (1972) Cyclic p-y Curve for Stiff Clay Above Water Table

3.2.7. Weak Rock (Reese)

Reese (1997) proposed empirical relationships for modeling lateral resistance of bored piles (drilled shafts) in rock layers with unconfined compressive strengths ranging from 10440 psf (500 kPa) to 104400 psf (5000 kPa). The p-y relationship is divided into three portions: initial slope (Eqn: 4.14a), transition (Eqn: 4.14b), and ultimate resistance (Eqn: 4.14c).

$$p = K_i y \quad for\ y \le y_a \tag{4.14a}$$

$$p = \frac{p_u}{2}\left(\frac{y}{y_m}\right)^{0.25} for\ y \ge y_a\ , p \ge p_u \tag{4.15b}$$

$p = p_u$,Otherwise (4.14c)

where

p = horizontal rock resistance per unit length

Ki = initial slope (initial modulus of subgrade reaction)

y = horizontal displacement

y_a = horizontal displacement at end of linear portion

p_u = horizontal ultimate rock resistance per unit length

$y_m = k_{rm} b$

k_{rm} = constant with values ranging from 0.0005 (more conservative) to 0.00005, that is used to establish the overall stiffness of the p-y relationship.

b = shaft diameter

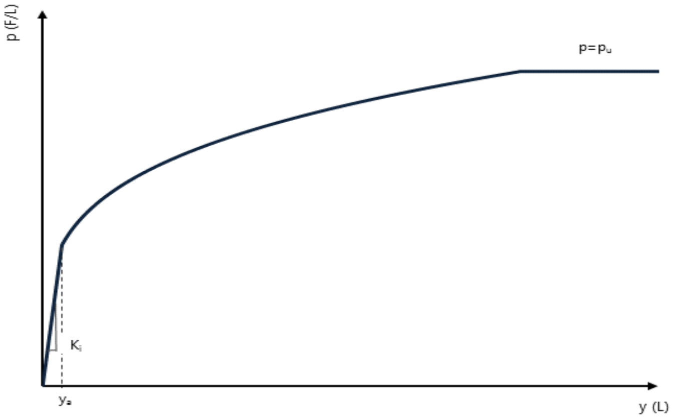


Figure 4.17 P-y relationship for weak rock (Reese, 1997)

The initial slope (modulus of subgrade reaction) is calculated as:

$$K_i = E_i \, k_i \quad (4.15)$$

where

Ei = initial rock mass modulus

ki = dimensionless constant

$$k_i = \left(100 - \frac{400zz}{3b}\right) for\ 0 \leq z \leq 3b \quad (4.16a)$$

$$k_i = 500\ for\ z > \ 3b \quad (4.16a)$$

Additionally, the horizontal displacement at end of linear portion (*ya*) is expressed as:

$$y_a = \left(\frac{p_u}{2k_i(y_m)^{0.25}}\right)^{1.333} \quad (4.17)$$

In turn, the ultimate resistance (*pu*) is expressed as:

$$q_u = \alpha_r q_u b\left(1 + 1.4\frac{z}{b}\right) \qquad for\ 0 \leq z \leq 3b \quad (4.18a)$$

$$q_u = 5.2\alpha_r q_u b \qquad for\ z > 3b \quad (4.18b)$$

where

αr = strength reduction factor

qu = unconfined compressive strength

z = depth below rock surface

The strength reduction factor varies linearly from 1.0 for rock quality designation (RQD)values of 0 to 0.333 for RQD values of 100%. Collectively, the above expressions lead to the following curve for horizontal resistance of weak rock.

3.3. Axial Soil Resistance

Axial soil modeling is comprised of side friction and tip resistance. Respective component forces are obtained from the following curves:

3.3.1. Axial t-z Curves for Side Friction

The following axial τ-z curves are available for modeling side friction:

3.3.1.1. Driven Piles (McVay)

McVay et al. (1989) recommended a t-z relationship to predict the load transfer through side resistance in driven piles:

$$z = \frac{\tau\, r}{G_i}\left[ln\left(\frac{(r_m-\beta)}{(r-\beta)}\right) + \frac{\beta(r_m-r)}{(r_m-\beta)(r-\beta)}\right] \quad (4.19)$$

where

τ = Side resistance (skin friction)

z = Vertical displacement (settlement)

r = Pile/Shaft radius

G_i = Initial (small-strain) shear modulus of soil

r_m = Outward radius were the transferred shear stress to soil is negligible

β = Side resistance parameter $\left(\beta = \frac{\tau\, r}{\tau_f}\right)$

τ_f = Maximun Shear stress

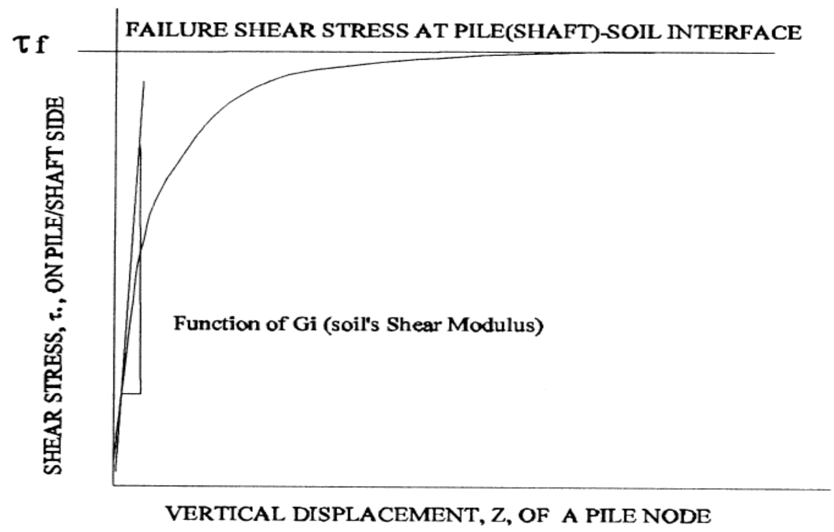


Figure 4.18 Axial τ-z Curves for Pile/Shaft

3.3.1.2. Drilled and Cast in situ Piles/Shaft

The τ-z curves used for drilled and cast insitu piles/shafts are based in the recommendations found in FHWA (1988). They are based in the trend lines and are computed for ach node. Trend line of stress transfer for axial end bearing and side resistance are provided for the for the following material:

Sand

Valid for $\varphi \geq 30^o$

$$\tau_u = K\sigma_z' \, tan\varphi = \beta\sigma_z' \leq 2.1 \, tsf(200 \, kPa) \quad (4.20)$$

$$\beta = 1.5 - 0.135z^{0.5}, \qquad 025 \leq \beta \leq 1.2 \quad (4.21)$$

where

τ_u = ultimate side resistance

K = parameter that combines lateral earth pressure and effects of installation

σ'_z = effective vertical stress at the depth in question

φ = Internal friction angle

β = dimensionless shaft friction factor

z = depth

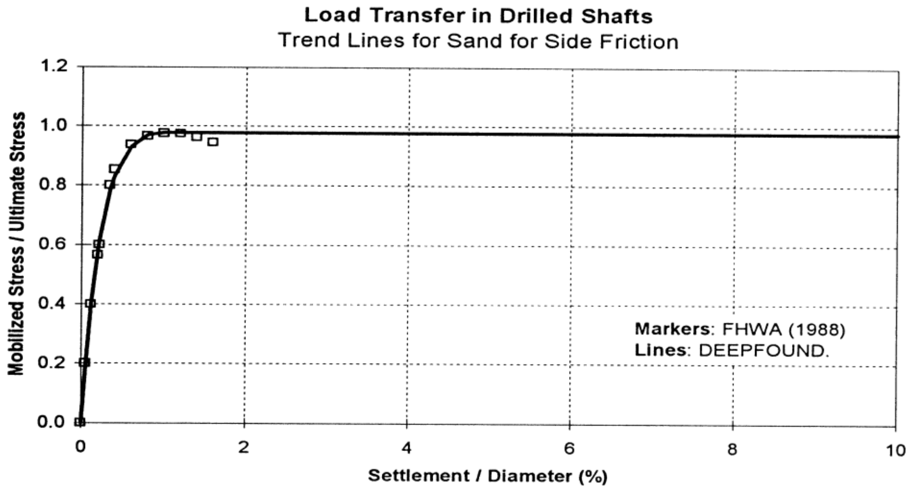


Figure 4.19 Trend Lines for Sand for Side Friction

Clay

$$\tau_u = \alpha c_{uz} \leq 2.75 \, tsf(263 \, kPa) \quad (4.22)$$

where

τ_u = Ultimate side resistance

c_{uz} = Undrained shear strength at depth z

α = Dimensionless shaft friction factor

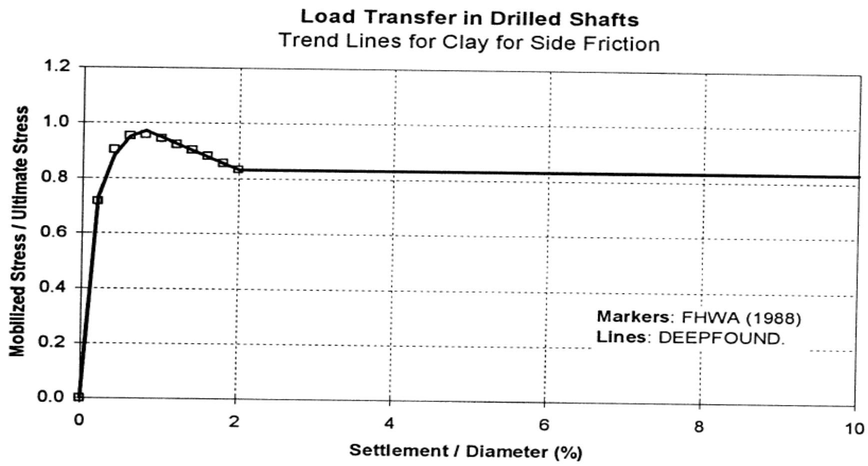


Figure 4.20 Trend Lines for Clay for Side Friction

3.3.1.3. Intermediate geomaterials (IGM)

Intermediate geomaterials (IGM) are characterized as one of the following 3 types:

Type 1: Argillaceous geomaterials: heavily overconsolidated clay, caly shale, saprolite and mudstone.

Type 2: Calcaeous Rock : Limestone and Limerock

Type 3: Very dense granular geomaterials : residul, completely decomposed rock and glacial till.

Note :

Types 1 and 2 are considered to be cohesive materials with an undrained strength (0.5 MPa < qu < 5 MPa.

Type 3 is primarily cohesionless and has Nspt from 50 to 100.

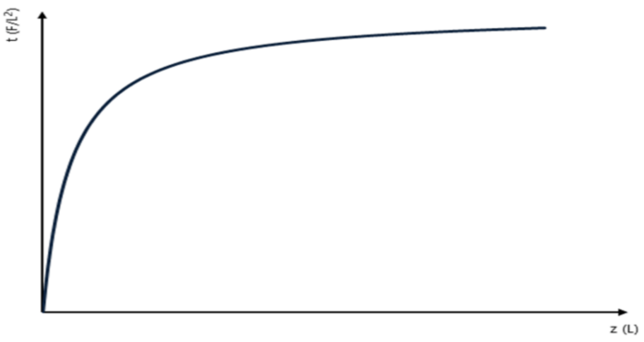


Figure 4.21 T-z curve for drilled shafts in cohesive IGM (O'Neill et al., 1996)

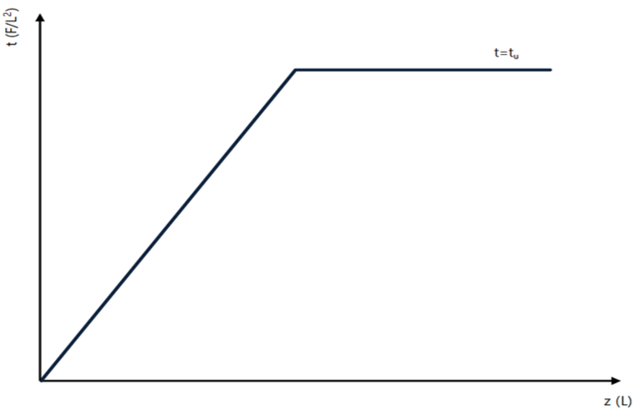


Figure 4.22 T-z curve for drilled shafts in non-cohesive IGM (Mayne and Harris, 1993)

3.3.2. Axial q-z Curves for Tip Resistance

Axial q-z curves for tip resistance are categorized for the followingg cases:

3.3.2.1. Driven Piles (McVay)

McVay et al. (1989) recommended a q-z relationship to predict the load transfer through end resistance in driven piles:

$$z = \frac{q(1-\nu)}{4r_b G_i\left(1-\frac{q}{q_u}\right)^2} \tag{4.23}$$

where

q = Axial end bearing (tip) resistance

z = Vertical displacement (settlement)

r_b = Pile radius at the base

G_i = Initial (small-strain) shear modulus of soil

q_u = Ultimate axial end bearing resistance

ν = Poisson's ratio of soil

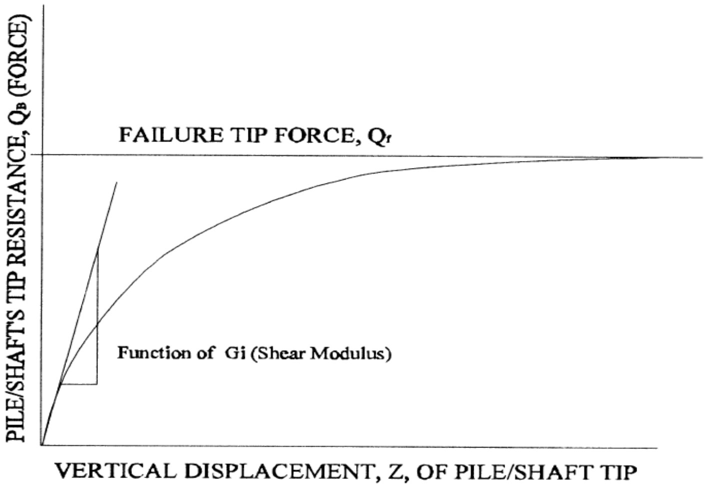


Figure 4.23 Axial q-z curve for driven pile

3.3.2.2. Drilled and Cast in situ Piles/Shaft

The q-z curves used for drilled and cast in-situ piles/shafts are based on recommendations found in Reese and O'Neill (1988) and Wang and Reese (1993).

Sand

Valid for N_{SPT} >10

Table 10 Representative values of qb

N_{SPT} (uncorrected)	q_b (tsf)	q_b (kPa)
0 - 75	0.60 N_{SPT}	57.5 N_{SPT}
> 75	45	4300

If B_b > 50 in (1.27 m): $q_{ur} = \frac{50}{B_b(in)} q_b = \frac{1.27}{B_b(m)} q_b$ (4.24)

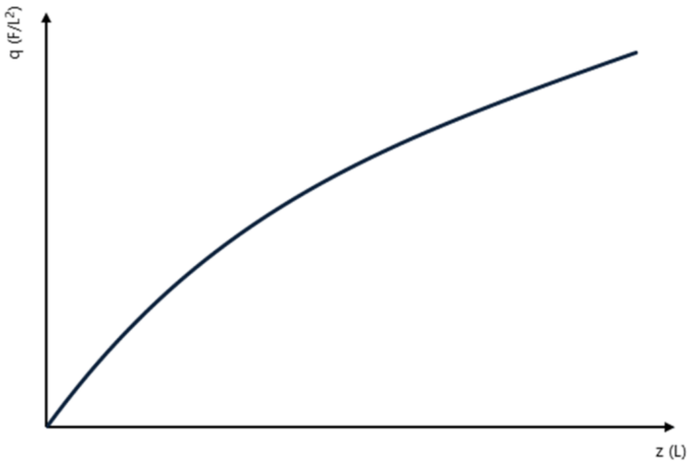


Figure 4.24 Q-z curve for drilled shafts in sand Reese and O'Neill (1988)

Clay

Ultimate load transfer in end bearing for drilled shafts in clay can be obtained through the following equation:

$$q_u = N_c c_{ub} \leq 40\ tsf (3830\ kPa) \qquad (4.25)$$

where

q_u = Ultimate end resistance

c_{ub} = Average undrained shear strength of the clay obtained 1-2 diameters below the shaft tip

N_c = End resistance factor

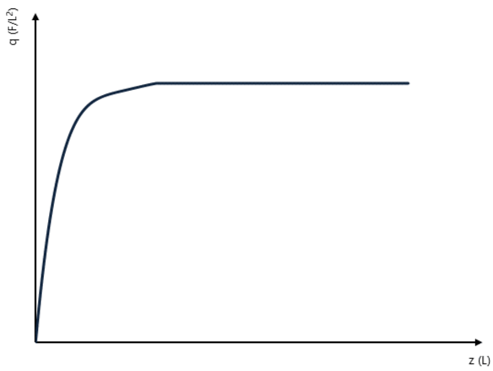


Figure 4.25 Q-z curve for drilled shafts in clay Reese and O'Neill (1988)

3.4. Torsional Soil Resistance

Torsional soil resistance is modeled using T-θ springs, where t is the torque applied to the pile and θ is the angle of twist, in radians. The springs are located at the nodal points. t-θ springs can be modeled as follows:

3.4.1. Hyperbolic Curve

Nonlinear torsional resistance of pile/shafts (T) against torque-induced rotation (twist angle, θ) is modeled using a hyperbolic T-θ curve, with initial slope as function of the shear modulus, G. the ultimate value is based on the ultimate shear at the contact pile/soil.

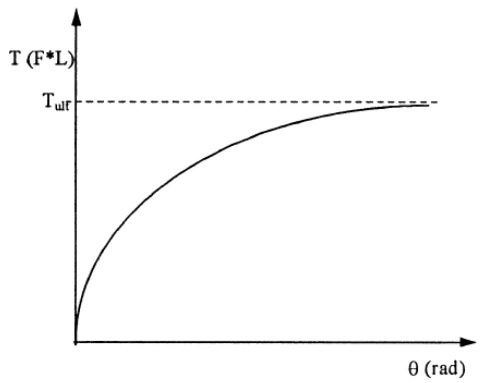


Figure 4.26 Hyperbolic representation of T- θ curve

For a length of pile ΔL, torque is given by

$$\Delta T = 2\pi r^2 \tau_0 \Delta L \tag{4.26}$$

where

ΔT = Torque increment

r = Pile radius

τ_0 = Shear stress along ΔL

ΔL = Pile length increment

Chapter 5

Results and Discussion

1 Project Data

1.1 Analysis of Pile Group

The FB-MultiPier program was used to analyze the behavior of the pile group foundation of the Bridge pier. The FB-MultiPier analyses were performed in order to predict the profiles of lateral displacement, shear forces, moment profiles and shear resistance force. The input parameters given in the FB-MultiPier for modeling pile and pile cap are summarized in Table 11; while the input parameters given for the pier are given in 12.

Table 11 Parameter of Material Properties of Pile and Pile Cap

Parameter	Pile	Pile Cap	Unit
Breadth (B)	3	11.2	m
Width (W)	3	7	m
Height (H)	25	3	m
Unit Weight (γ_c)	24	24	kN/m^3
Elastic Modulus (E_c)	28958	28958	(MPa)
Poisson's Ratio (v)	0.2	0.2	
Thickness(t)		3	m
Compressive Strength (f'_c)	34.474	34.474	(MPa)

Table 12 Geometry of Pier Column and Pier Cap

<table>
<tr><th rowspan="2">Pier Height H_c (m)</th><th rowspan="2">Column offset L_1 (m)</th><th rowspan="2">Column spacing L_2 (m)</th><th rowspan="2">Cantilever length L_3 (m)</th><th rowspan="2">Pier Column (no.)</th><th colspan="2">Column Section</th><th colspan="2">Pier Cap section</th></tr>
<tr><th>Width (m)</th><th>Depth (m)</th><th>Width (m)</th><th>Depth (m)</th></tr>
<tr><td>9.5</td><td>3</td><td>5</td><td>4.5</td><td>2</td><td>1.5</td><td>1.5</td><td>1.5</td><td>1.2</td></tr>
</table>

Table 13 Parameter of Material Properties of Pier Column and Pier Cap

Material Type	Parameter	Number	Unit
Mild Steel	Yield Stress	413.7	(MPa)
	Young's Modulus	200000	(MPa)
	Poisson's Ratio (*v*)	0.3	
Concrete	Concrete Modulus (E_c)	28958	(MPa)
	Poisson's Ratio (*v*)	0.2	
	Compressive Strength (f'_c)	34.474	(MPa)

Table 14 Parameter for Modeling Soil Layers.

Depth (m)		Layer	Soil	Unit Weight γ_t (kN/m^3)	Undrained Shear Strength C_u (kPa)	Friction Angle φ (deg)	Subgrade Modulus k (kN/m^3)	Strain ε_{50}
- 4.6	-7	1	Soft to very soft clay	17.28	10.3		35830	0.025
-7	-9.1	2	Fine Sand	17.60		30	16830	
-9.1	-13,7	3	Silty Soft Clay	18.85	22		38000	0.015
-13.7	-17.4	4	Silty Sand	18.54		34	20090	
-17.4	-22.3	5	Stiff Clay	**17.91**	52.7		40720	0.01

Depth (m)		Layer	Soil	Unit Weight γ_t (kN/m^3)	Unconfined compressive Strength q_u (MPa)	Mass Modulus E_m (MPa)	RQD %	E_m/E_i	Surface Rough = 1 Smooth = 2	Split Tension Strength (MPa)	Unit Weight γ_c (kN/m^3)	Slump (cm)	K_{rm}
-22.3	-30	6	Weak Rock	22	2.4	137.8	20	0.5	1	1.93	24	15	0.0005

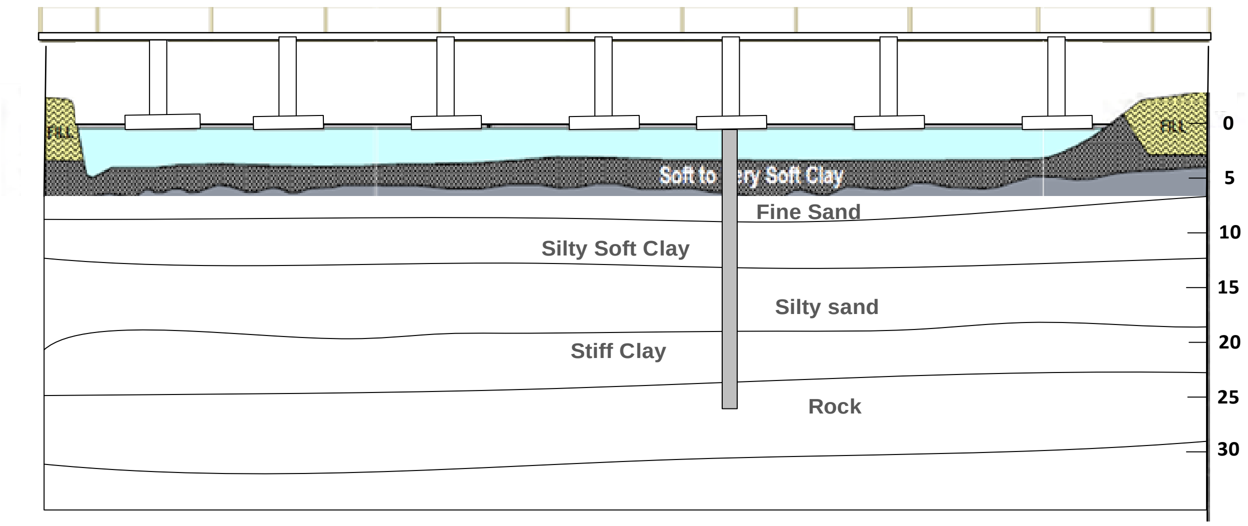


Figure 5.1 Generalized subsurface profile at span bridge

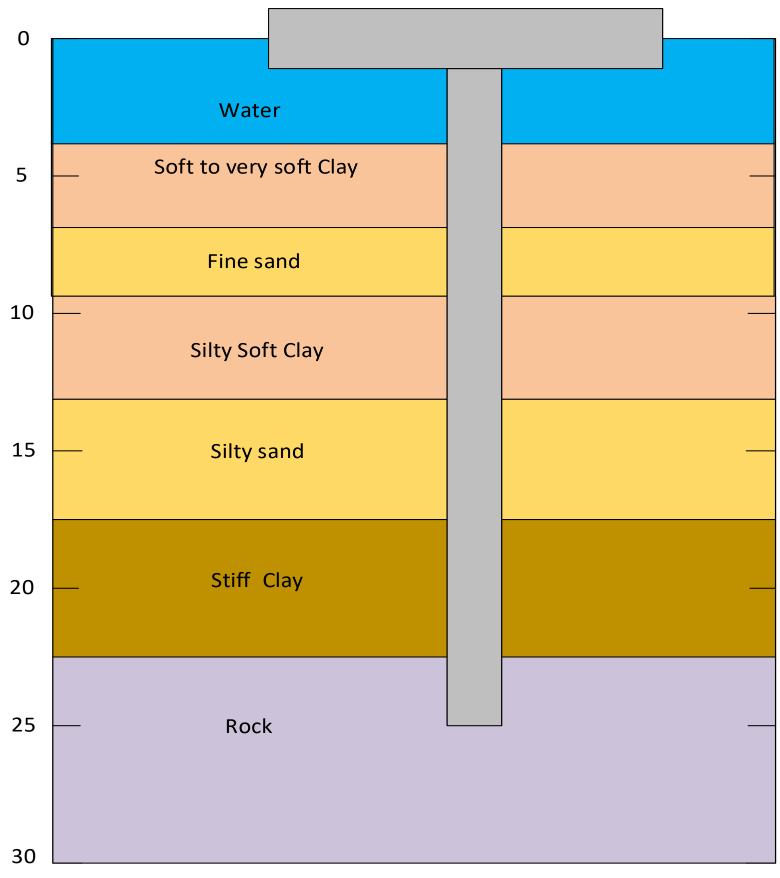


Figure 5.2 Soil Layers Profile

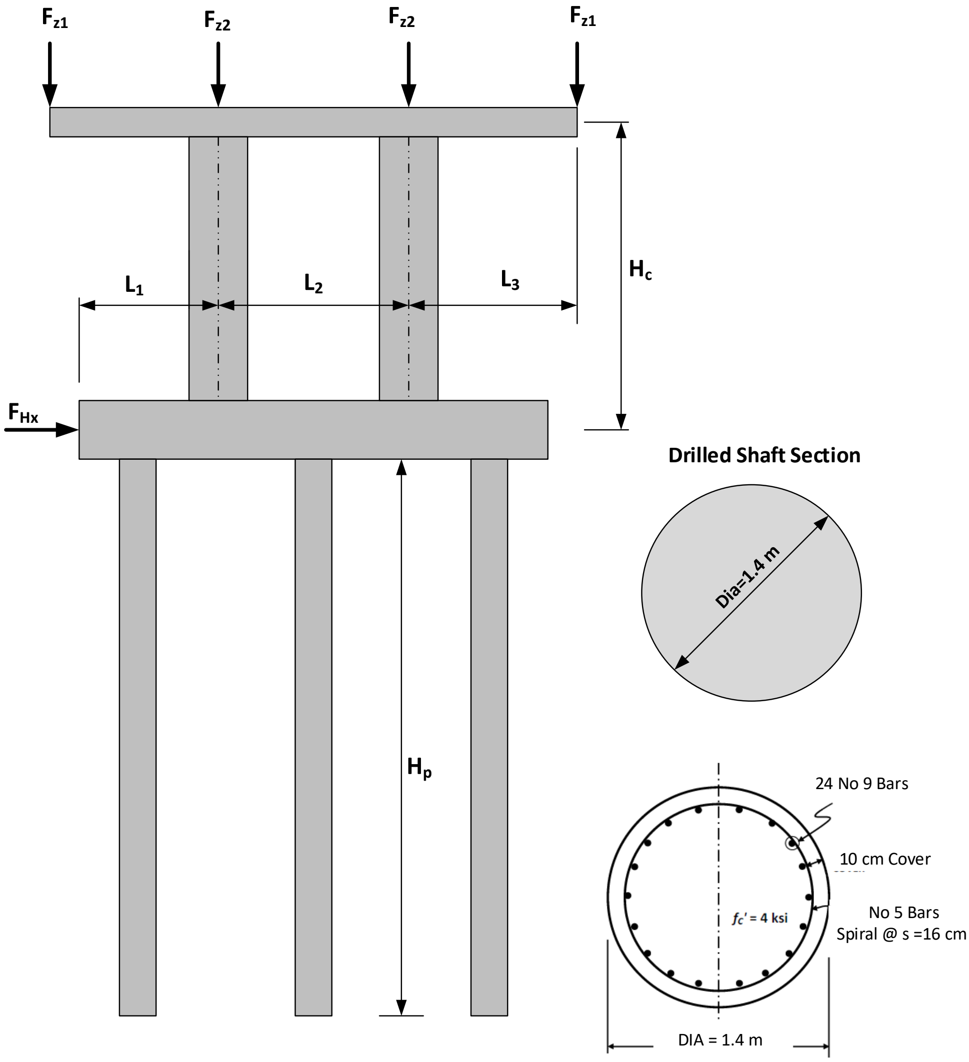


Figure 5.3 Bridge Pier Structure

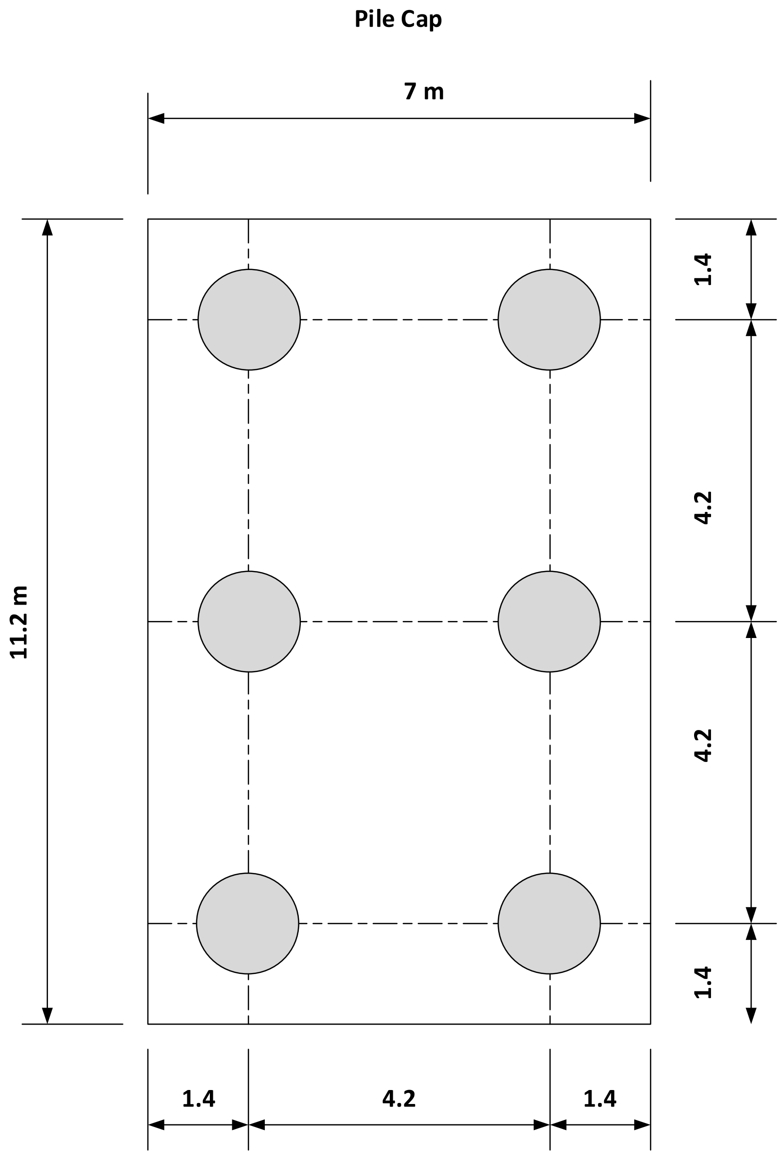


Figure 5.4 Pile Plan View –Pile Section

Table 15 Loads on Bridge Pier Structure

Load	Number	Value	Unit
F_{z1}	2	670	kN
F_{z2}	2	1120	kN
F_{H1}	1	4450	kN

2 Results

2.1 SOIL LAYER MODELS

2.1.1 Lateral loaded Pile

Lateral behavior is defined by a series of p-y curves that define the lateral deflection a pile experiences under a certain amount of load.

P-Y Curves

The P-Y curves profile were predicted by the FB-Multipier. P-Y curves represent the lateral response of piles under loading and are influenced by factors such as soil type, loading conditions, pile-soil interaction, and depth. These curves provide information about the relationship between lateral soil resistance (P) and lateral deflection (Y) at different depths along the pile.

The first step in constructing P-Y curves is to determine the soil type for each layer. In this case, the first layer is described as a cohesive soil known as "Clay soil". For this case , The Matlock method, (1970), is used to model the lateral behavior of loaded piles. The P-Y curves obtained using the Matlock method for the first layer (clay soil) include information about the ultimate load at both the bottom and top of the layer. The ultimate load represents the maximum lateral load that the pile can sustain before failure. Figure 5.5 presents the relationship between lateral load and lateral deflection at different depths within the layer.

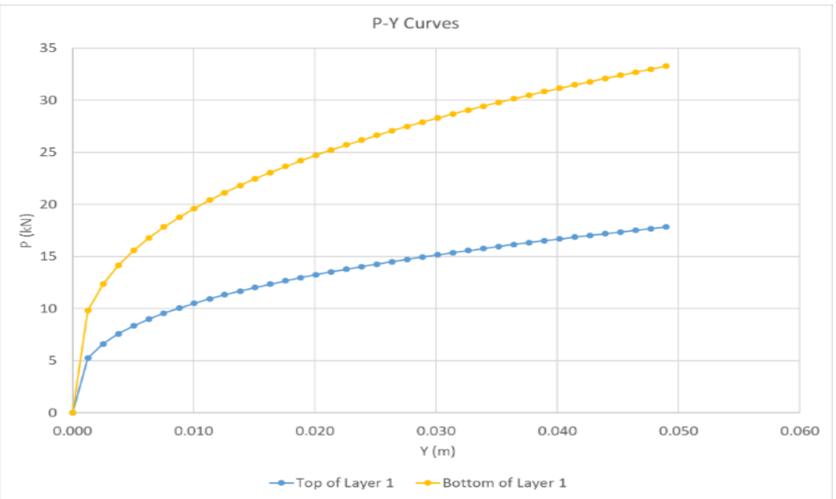


Figure 5.5 P-Y Curves Layer 1 Profile

The P-Y curves for layer 2 were generated using the FB-MultiPier program. Layer 2 is described as a cohesionless soil, specifically "Fine Sand", and been analyzed using Reese method. The curves obtained for this layer 2 showed that the response of the pile at the bottom of layer 2 differs from that at the top. Within the P-Y curves for layer 2, there are three distinct points. These points likely represent key locations or stages in the lateral behavior of the pile in the sand soil (Reese method). Between two of the distinct points, the P-Y relationship is described as linear. Between the remaining points, the P-Y relationship is described as parabolic (Figure 5.6). This suggests a curved relationship between lateral load and deflection, which is characteristic of the behavior of the pile in the sand soil.

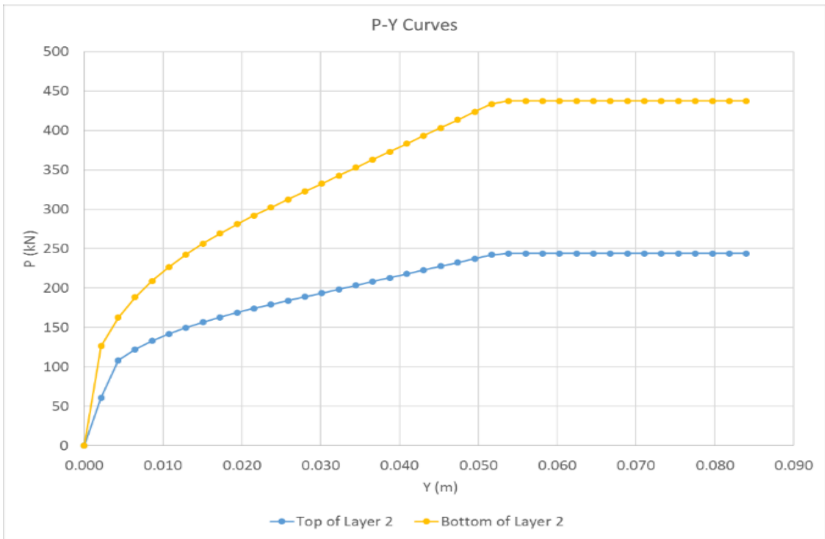


Figure 5.6 P-Y Curves Layer 2 Profile

The P-Y curves for layer 3 were generated using the FB-MultiPier program. In this case, the 3rd layer is described as a cohesive soil known as "silty soft Clay". For this case , The Matlock method, (1970), is used to model the lateral behavior of loaded piles. The P-Y curves obtained using the Matlock method for this layer include information about the ultimate load at both the bottom and top of the layer. Figure 5.7 presents the relationship between lateral load and lateral deflection at different depths within the layer.

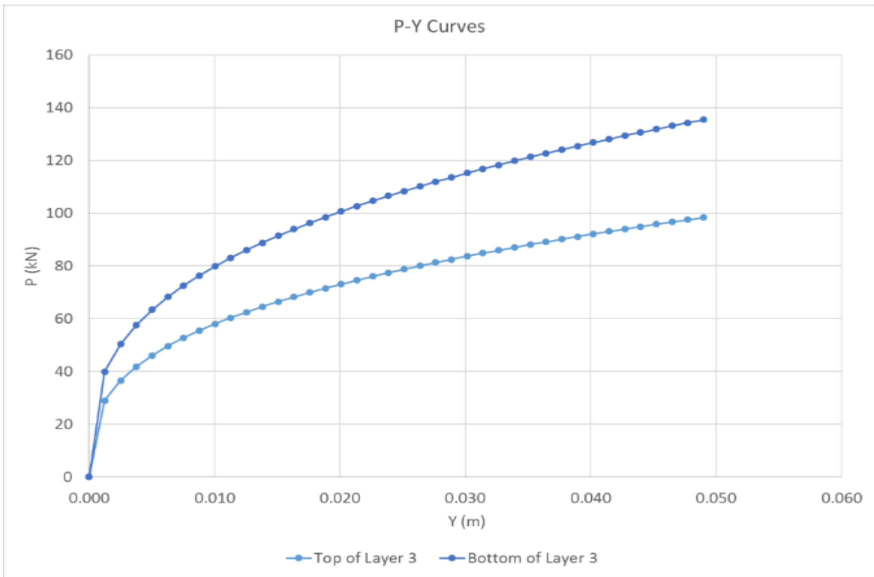


Figure 5.7 P-Y Curves Layer 3 Profile

The P-Y curves for layer 4 were generated using the FB-MultiPier program. Layer 4 is described as a cohesionless soil, specifically "Silty Sand", and been analyzed using Reese method (1974). The curves obtained for this layer showed that the response of the pile at the bottom of layer 4 differs from that at the top. Within the P-Y curves for layer 2, there are three distinct points. Between the two of the distinct points, the P-Y relationship is described as linear. Between the remaining points, the P-Y relationship is described as parabolic (Figure 5.8). This suggests a curved relationship between lateral load and deflection, which is characteristic of the behavior of the pile in the sand soil.

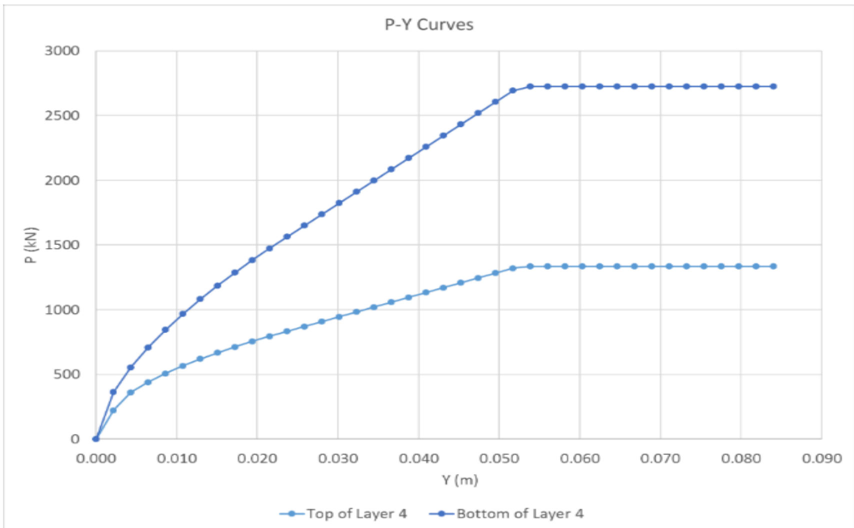


Figure 5.8 P-Y Curves Layer 4 Profile

The P-Y curves generated by the FB-MultiPier program determine the effective stress of the soil at a specific depth. In this case, layer 5 is described as a cohesive soil, specifically "Stiff

Clay." and For this case , The Matlock method (Stiff with free water1975), is used to model the lateral behavior of loaded piles. Considering the lateral loaded pile models and using the soil type information for each layer, the P-Y curve for the top and bottom of layer 5 are the same. This implies that both the top and bottom of layer 5 exhibit similar behavior in terms of the relationship between lateral load and lateral deflection. Figure 5.9 likely presents the P-Y curves for layer 5, The curves provide information on the lateral response of the pile in the cohesive clay soil at different depths within layer 5.

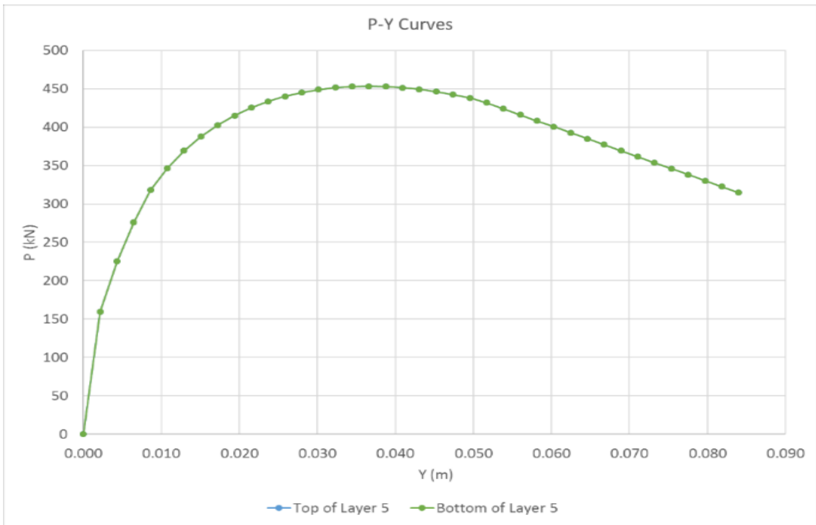


Figure 5.9 P-Y Curves Layer 5 Profile

As it is previously mentioned, the FB-MultiPier program is used to generate P-Y curves, which illustrate the influence of specific soil parameters on the lateral deflection of piles. In this case, the program is applied lower layer soil, which is rock layer described as "Weak Rock." The lateral loaded pile models, considering the Reese method, are used to analyze the pile behavior in weak rock. The P-Y curve shown in Figure 5.10 represents the relationship between lateral load and lateral deflection for the top layer.

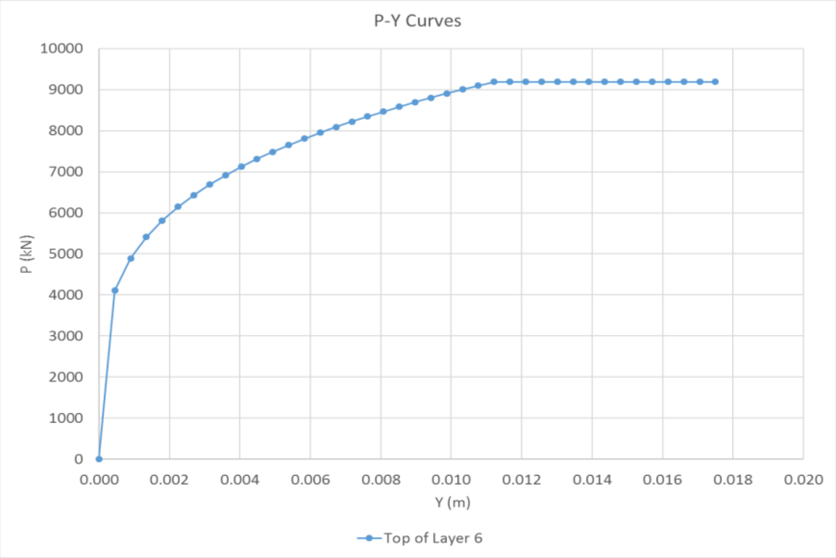


Figure 5.10 P-Y Curves Layer 6 Profile

2.1.2 Axial loaded Pile (Side Friction)

The axial behavior is defined by the type of structural element analyzed. T-z curves that define the amount of axial force required to move the structural element in a downward direction model the soil behavior during the installation process of the structural element. The axial soil structure interaction is accomplished with nonlinear axial, T-z, springs acting along the length of each pile. A typical axial T-z curve for side friction used in FB-Pier were assigned based on soil layering presented in Figures 5.11 to 5.16. The model used for all soil layers in the profile was that for drilled shaft.

T-Z Curves

FB-MultiPier program generates axial soil response curves (T-z curves) for the for layer 1 using the soil type information (soft to very soft Clay) and applying the Drilled Shaft Clay Model. As seen in Figure 5.11, these curves present the axial load (side friction) versus depth relationship and provide valuable information about the behavior of the cohesive clay soil surrounding the drilled shaft.

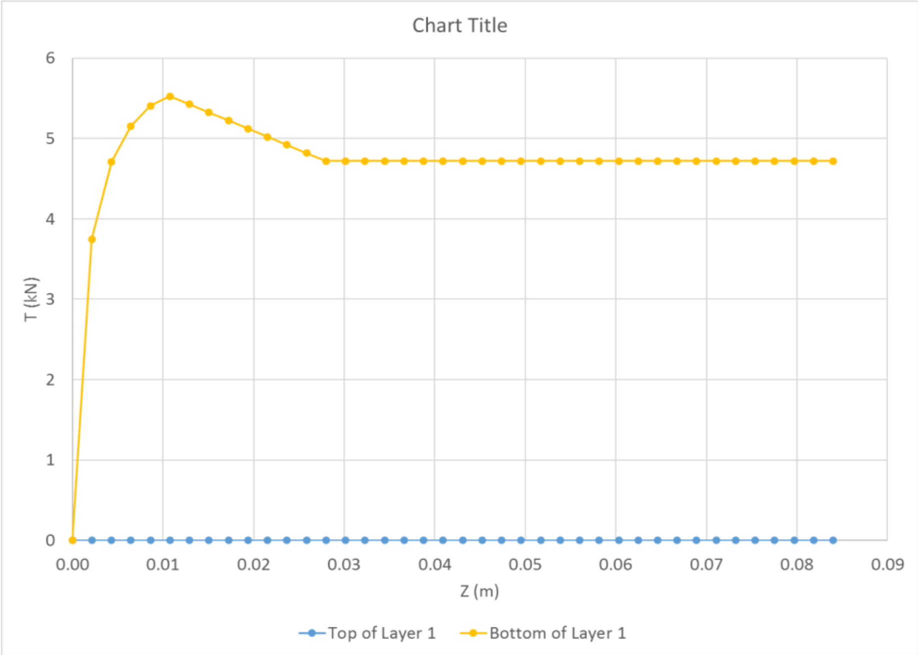


Figure 5.11 T-Z Curves Layer 1 Profile

Axial soil modeling is comprised of side friction, were performed using the FB-MultiPier program, for the for layer 2 using the soil type information (Fine Sand) and applying the Drilled Shaft Sand Model. As showed in Figure 5.12, these curves present the axial load (side friction) versus depth relationship and provide valuable information about the behavior of the cohesionless sand soil surrounding the drilled shaft.

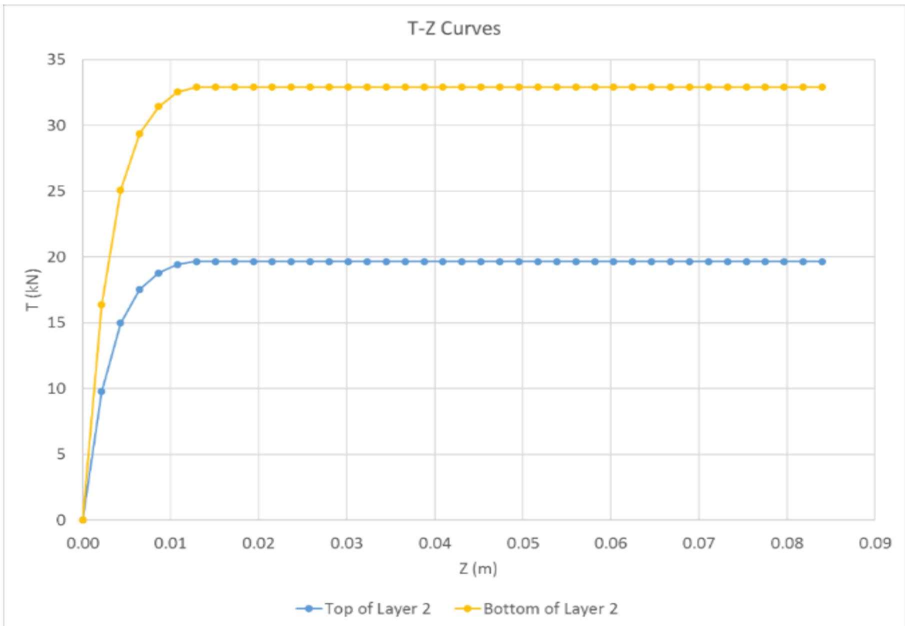


Figure 5.12 T- Z Curves Layer 2 Profile

FB-MultiPier program generates axial soil response curves (T-z curves) for the for layer 3 using the soil type information (silty soft Clay) and applying the Drilled Shaft Clay Model. As seen in Figure 5.13, these curves present the axial load (side friction) versus depth relationship and provide valuable information about the behavior of the cohesive clay soil surrounding the drilled shaft.

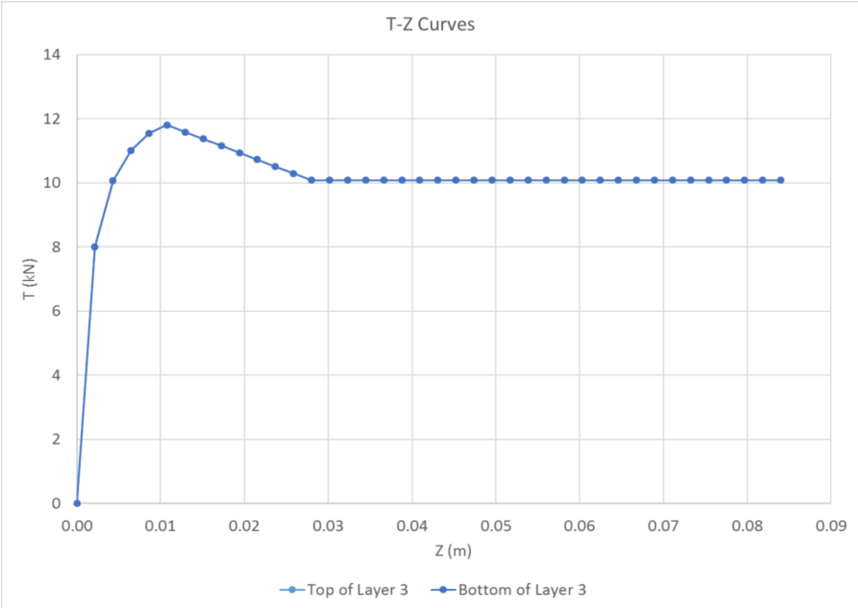


Figure 5.13 T- Z Curves Layer 3 Profile

Axial soil modeling is comprised of side friction, were performed using the FB-MultiPier program, for the for layer 4 using the soil type information (Silty Sand) and applying the Drilled Shaft Sand Model. As seen in Figure 5.14, these curves present the axial load (side friction) versus depth relationship and provide valuable information about the behavior of the cohesionless sand soil surrounding the drilled shaft.

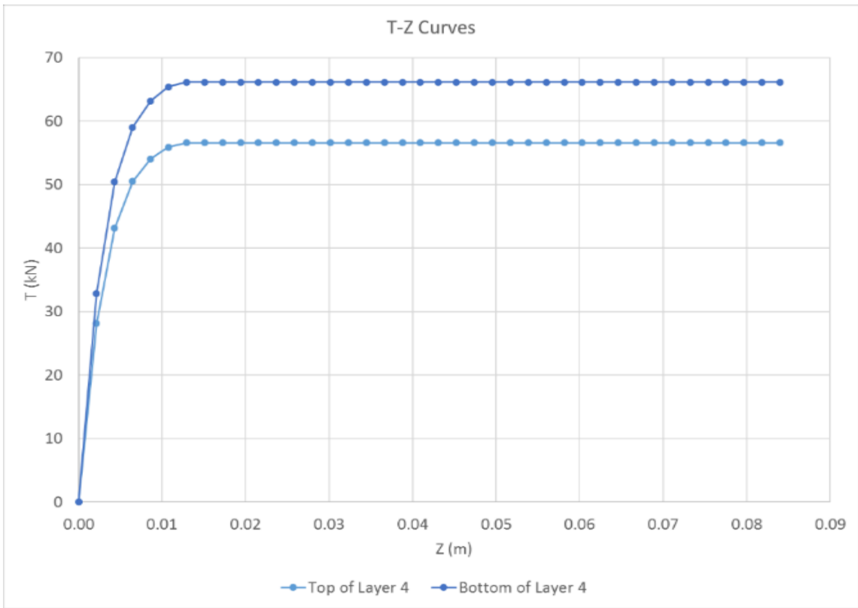


Figure 5.14 T- Z Curves Layer 4 Profile

Axial soil response curves (T-z) obtained from FB-MultiPier, for the for layer 5 using the soil type information (stiff Clay) and applying the Drilled Shaft Clay Model. As showed in Figure 5.15, these curves present the axial load (side friction) versus depth relationship and provide valuable information about the behavior of the cohesive clay soil surrounding the drilled shaft.

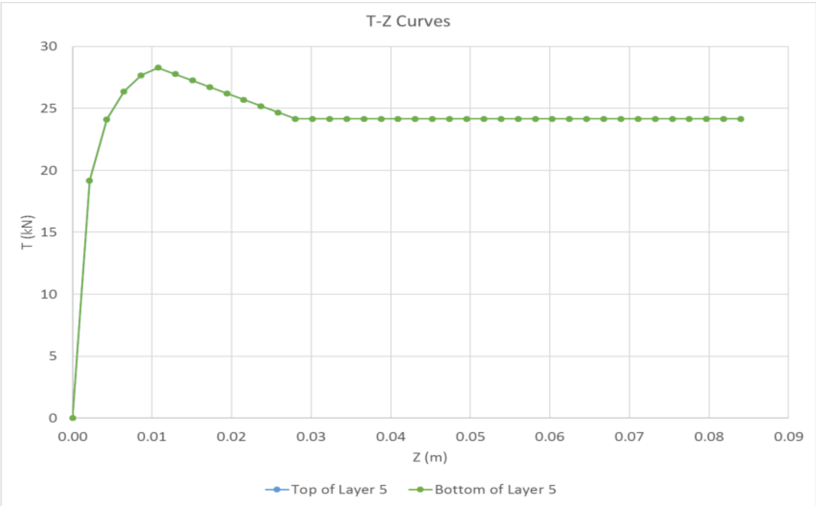


Figure 5.15 T- Z Curves Layer 5 Profile

As it is previously mentioned, the FB-MultiPier program is used to generate T-z curves. In this case, the program is applied lower layer soil, which is rock layer described as "Weak Rock". The axial loaded pile models, considering the Reese method, are used to analyze the pile behavior in weak rock. The t-z curve shown in Figure 5.16 represents the relationship between shear stress and displacement of the soil-pile interface at a depth for the top layer

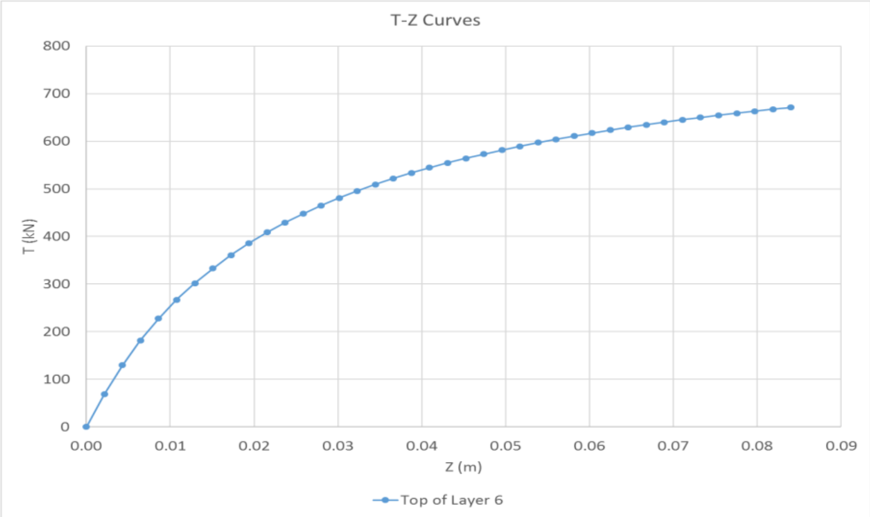


Figure 5.16 T- Z Curves Layer 6 Profile

2.1.3 Axial loaded Pile (Tip)

The tip model reflects the type of structural element analyzed. The built in Q-z models are used to simulate the installation process of the selected structural element.

Q-Z Curves

Axial soil modeling in the FB-MultiPier program includes the consideration of tip resistance. This means that the response of the soil at the tip of the pile, also known as the end bearing capacity, is taken into account in the analysis.The tip resistance acting on the bottom of each pile (Shaft) was modeled with an axial Q-z curve based on bearing properties of layer 6. Figure 5.17 illustrates the Q-z curves obtained from the FB-MultiPier program, showcasing the axial soil response for the pile. The curves provide information about the relationship between axial load and depth.

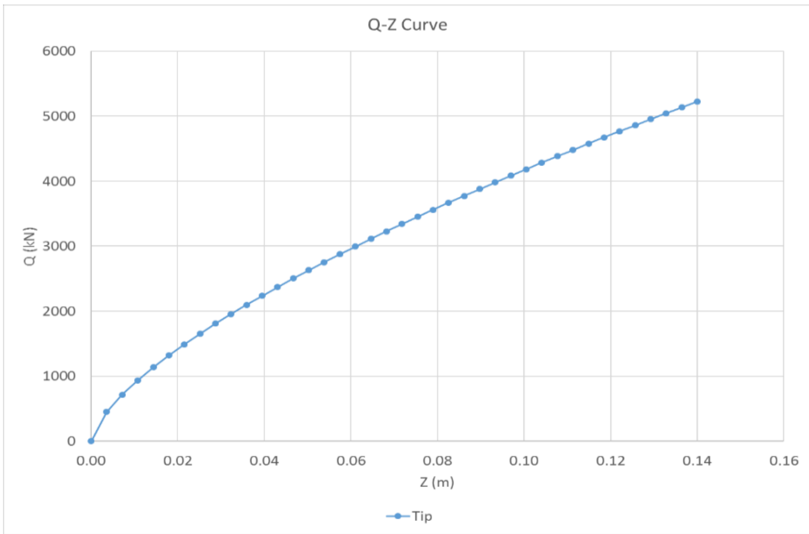


Figure 5.17 Q- Z Curve Tip Profile

2.2 PILE RESULTS

Results can be viewed using FB-Multipier, after reaching a converged solution to the problem. The pile results is view the maximum and minimum results for the piles. The load combinations with load are most likely to produce the maximum and minimum results. When the analysis is completed, this will highlight the pile with the demand/capacity ratio, show the force results, the moment and pile displacements. The resulting plots of linear and non-linear behavior are showed in Figures 5.18 and 5.19.

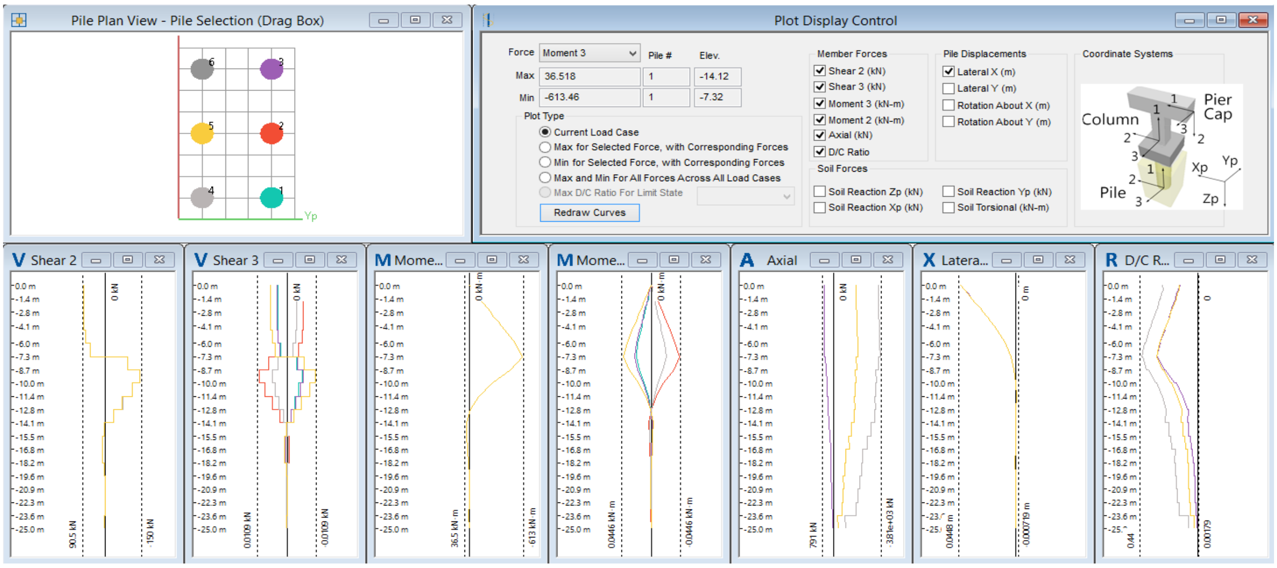


Figure 5.18 FB-Multipier Screenshot for Linear Pile Results

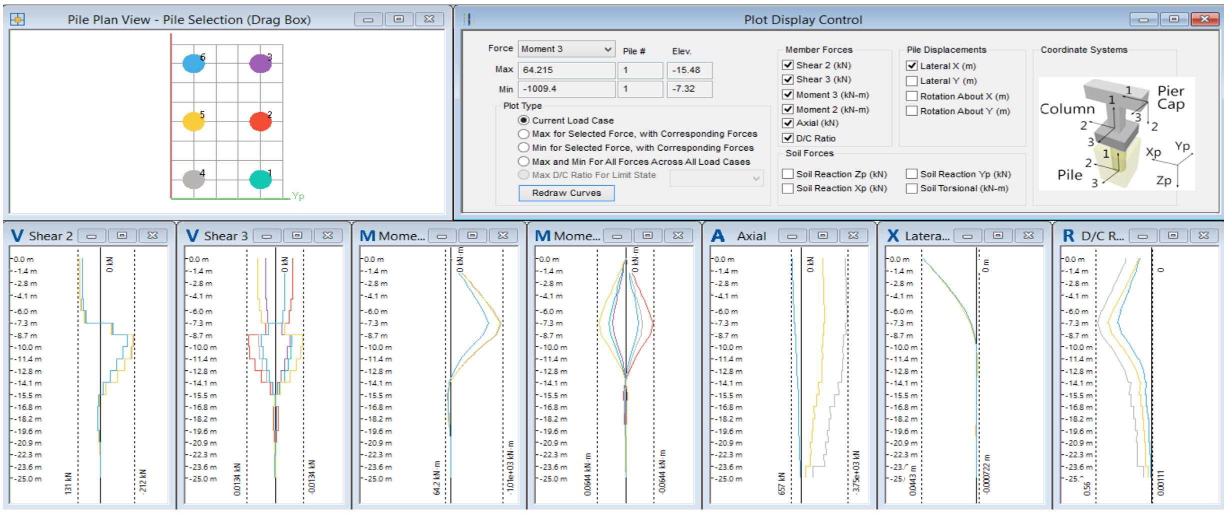


Figure 5.19 FB-Multipier Screenshot for Non-Linear Pile Results

2.2.1 DISCUSSION

2.2.1.1 Axial Force Profiles

The profile of axial force developed in piles when applying loading were determined from the FB-MultiPier analysis. The axial force profiles developed along the piles located in different rows at the applied load. FB-Multipier showed that the piles nears the loading zones develop tensile forces and those located farther from the loading zone develop compressive forces.

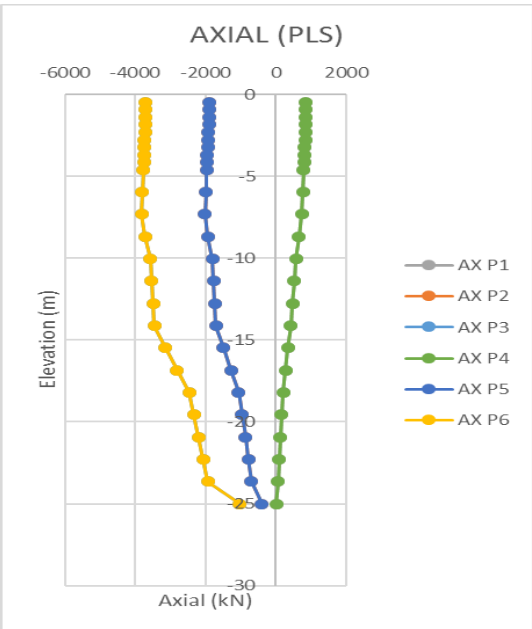


Figure 5.20.1 Linear Axial Force Profile

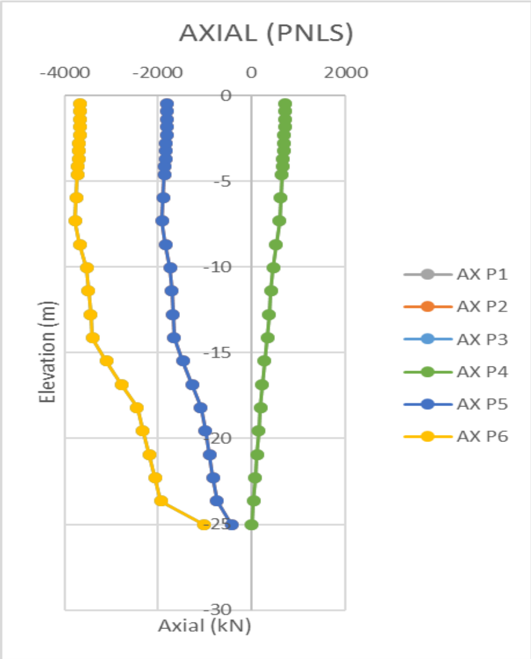


Figure 5.20.2 Non-Linear Axial Force Profile

The values of predicted maximum axial force considering the linear material analysis of the piles are 856 kN (tension) and 3700 kN (compression) corresponding to the static loads (Figure 5.20.1). Similarly, the values of maximum axial force predicted by the FB-MultiPier considering the non-linear material analysis of the piles are 702 kN (tension) and 3670 kN (compression) corresponding to applied lateral loads (Figure 5.20.2). It is observed that the axial force values predicted form the FB-MultiPier analyses using both linear and non-linear material behavior of pile have similar trend and shape at all applied loads, but the non-linear analysis produce about 10% lower value of axial force as shown in Figure 5.20.3.

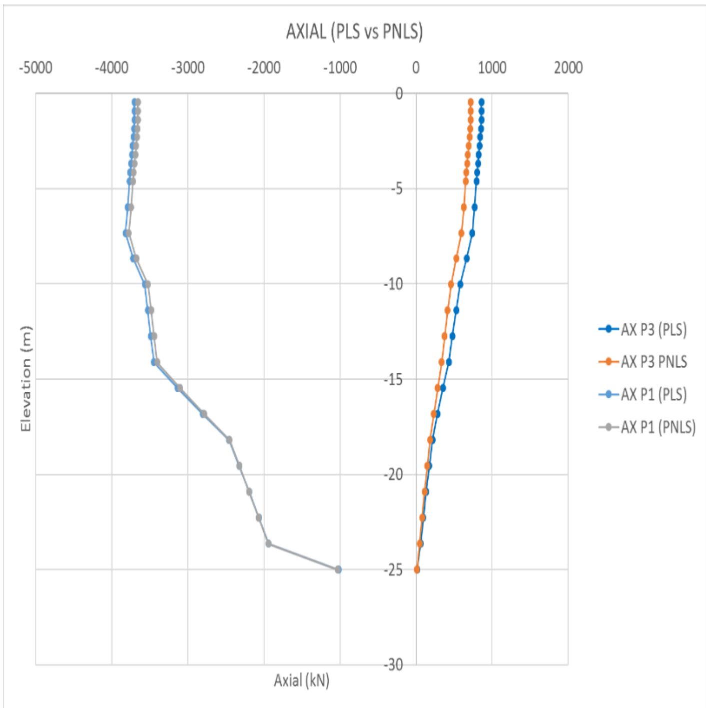


Figure 5.20.3 Linear vs Non-Linear
Axial Force Profile

2.2.1.2 Shear Force Profiles

The profiles of shear force were performed using the FB-MultiPier program. The profiles of shear force predicted by FB-MultiPier assuming both linear and non-linear material behavior of the piles are presented in Figures 5.21.1 and 5.21.2. The shear force predicted for all piles are very close and have similar trend and shape for both behaviors. In both cases, the predicted value of shear force is almost between 91 kN(min) to 94 kN (max) and between 130 kN(min) to 135 kN(max) at the pile head respectively and remain constant till the depth 4.6 m. The predicted shear force then gradually reduced to zero at 7.5 m depth. The maximum predicted negative shear force predicted corresponding to all applied lateral load is found at 10 m depth below the pile head, and the value is almost between 148 kN(min) to 151 kN(max) and between 179 kN(min) to 182 kN(max). It can be observed that the depth to the zero shear force occurred between 15-25 m from pile top for all 6 piles. Therefore the non-linear analysis produce higher value of shear force as seen in Figure 5.21.3.

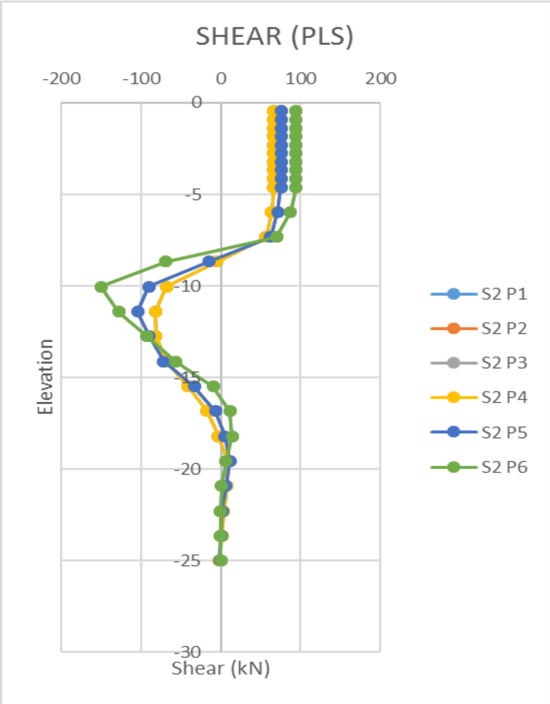


Figure 5.21.1 Linear Shear Force Profile

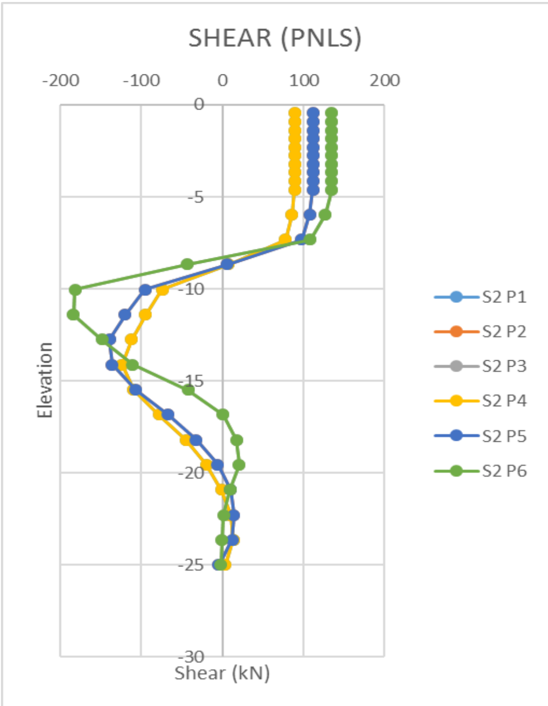


Figure 5.21.2 Non-Linear Shear Force Profile

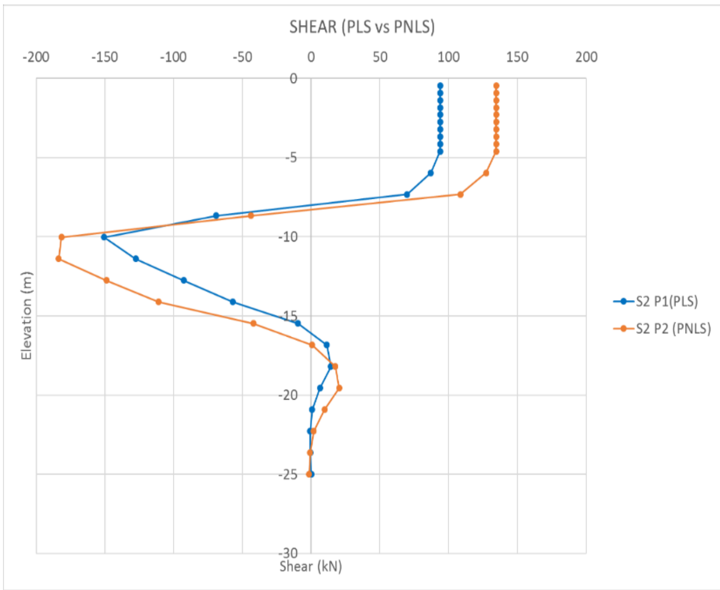


Figure 5.21.3 Linear vs Non-Linear **Shear Force Profile**

2.2.1.3 Bending Moment Profiles

The moment profiles of the vertical piles were achieved by the FB-MultiPier analysis assuming both linear and non-linear material behavior of the piles. These moments predicted by the FB-MultiPier linear and non-linear analyses at the static lateral loads were determined and presented in Figures 5.22.1 and 5.22.2 respectively.

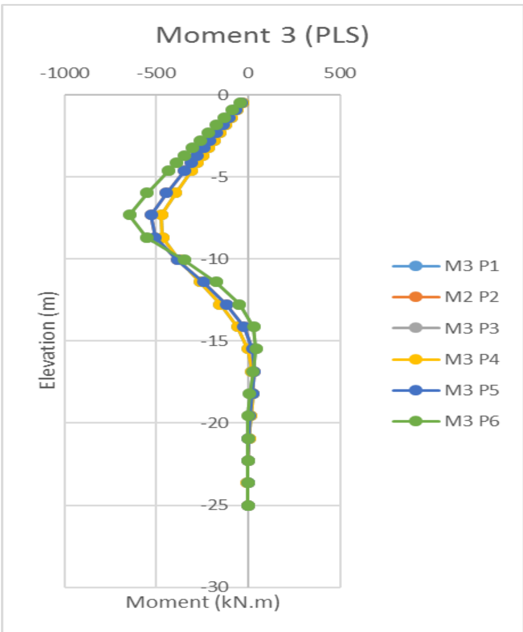


Figure 5.22.1 Linear Bending Moment Profile

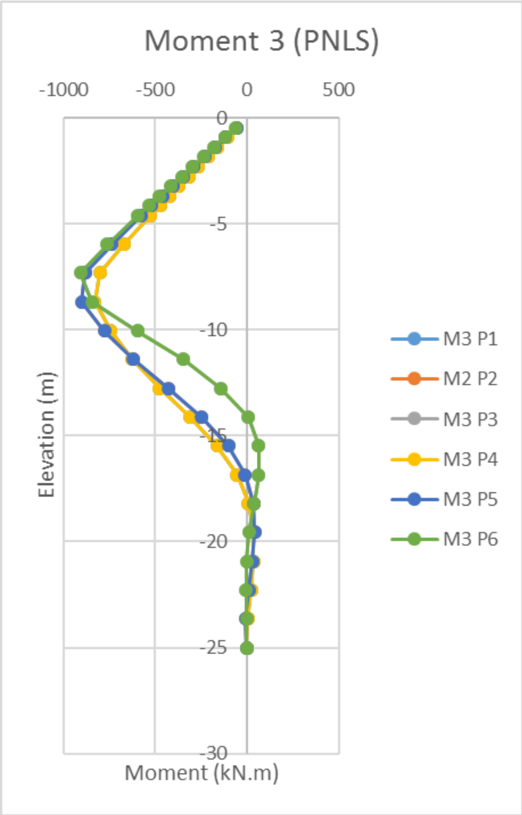


Figure 5.22.2 Non-Linear Bending Moment Profile

The curves of load versus maximum moments observed from the linear analysis using FB-MultiPier are compared with the non-linear analysis as shown in Figure 5.22.3. The curve clearly shows that the predicted moments from the FB-MultiPier are very close to each other.

In the linear analysis, the profiles of predicted moment force have similar trend and shape at all applied loads for all piles and the maximum moment is about 645 kN.m. In the non-linear analysis, the profiles of predicted moment force have similar trend and shape at all applied loads for all piles and the maximum moments by FB-MultiPier is 905 kN.m. Therefore the non-linear analysis produce Higher value moment as seen in Figure 5.22.3.

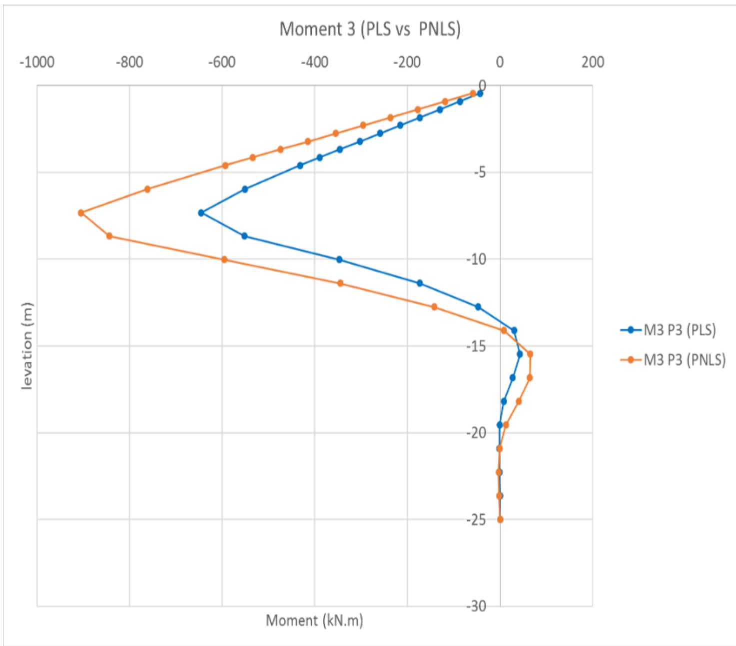


Figure 5.22.3 Linear vs Non-Linear **Bending Moment Profile**

2.2.1.4 Soil Resistance Profiles

The profiles of soil resistance were predicted by the FB-MultiPier considering both the linear and non-linear material behavior of pile and the results are presented in Figures 5.23.1 and 5.23.2 respectively. The values of predicted maximum soil resistance considering the linear material analysis of the piles are 139 kN corresponding to the static lateral loads. Similarly, the values of maximum soil resistance predicted by the FB-MultiPier considering the non-linear material analysis of the piles are 153 kN corresponding to applied lateral loads. It is observed that the lateral soil resistance values predicted form the FB-MultiPier analyses using both linear and non-linear material behavior of pile have similar trend and shape at all applied loads for all piles, but the non-linear analysis produce about 10% higher value of soil resistance as shown in Figure 5.23.3.

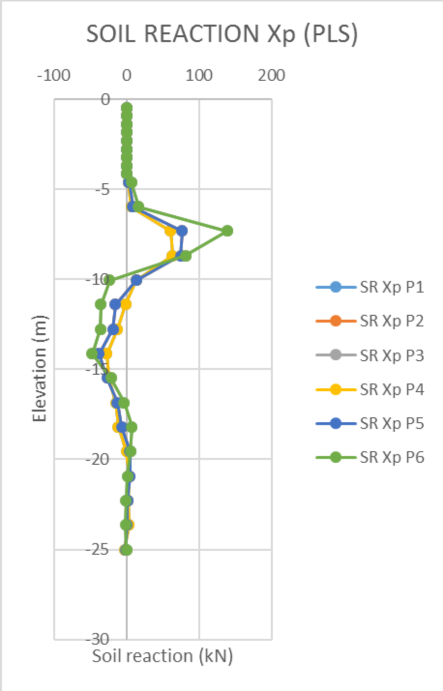


Figure 5.23.1 Linear Soil Reaction Xp Profile

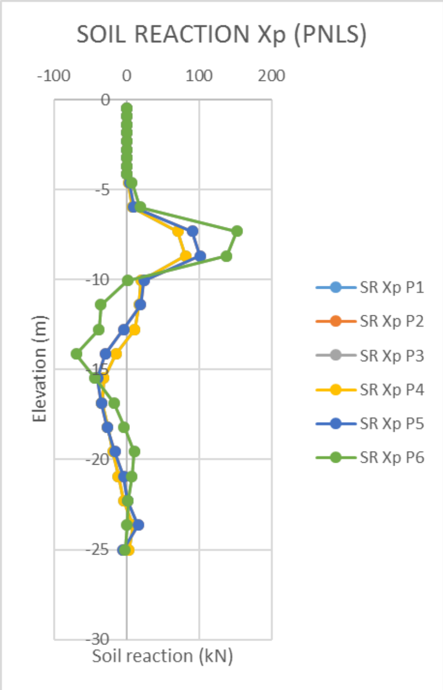


Figure 5.23.2 Non-Linear Soil Reaction Xp Profile

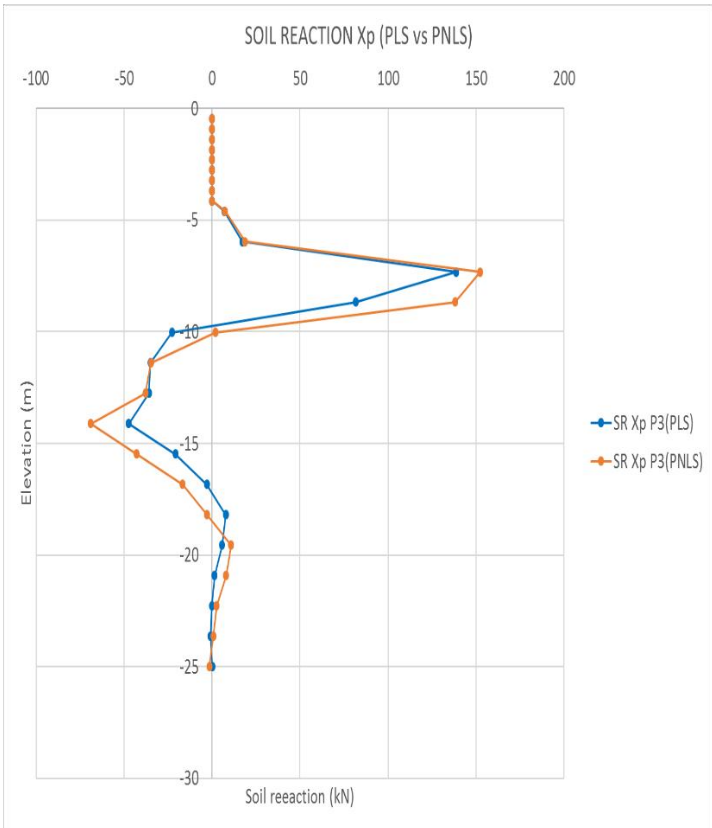


Figure 5.23.3 Linear vs Non-Linear
Soil Resistance Profile

2.2.1.5 Lateral Displacement Profiles

The lateral displacement profiles of the piles were accomplished using the FB-MultiPier analysis by assuming both linear and non-linear behavior of pile material. The analysis was determined at the static lateral loads. The resulting predicted lateral displacement profiles are presented in Figures 5.24.1 and 5.24.2 respectively.

The FBMultiPier predicts the maximum lateral displacements at pile head of 0.047 m and 0.055 m in both linear and non-linear case respectively. It is observed that the lateral displacements does not change for all piles in each case as shown (Figure 5.24.3). but the non-linear analysis produce 15% larger value of lateral displacement as presented in Figure 5.24.3.

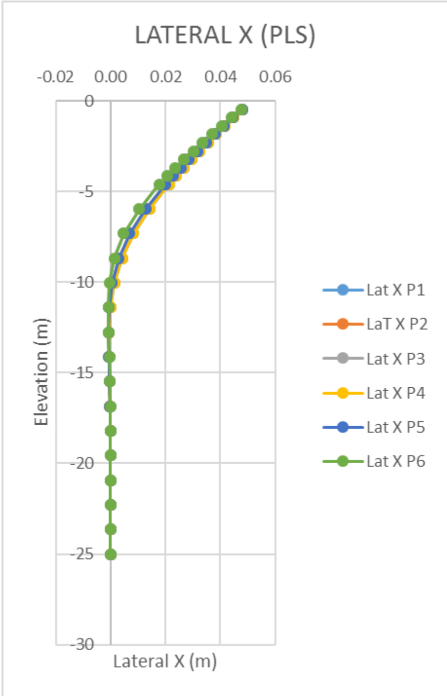


Figure 5.24.1 Linear Lateral X Profile

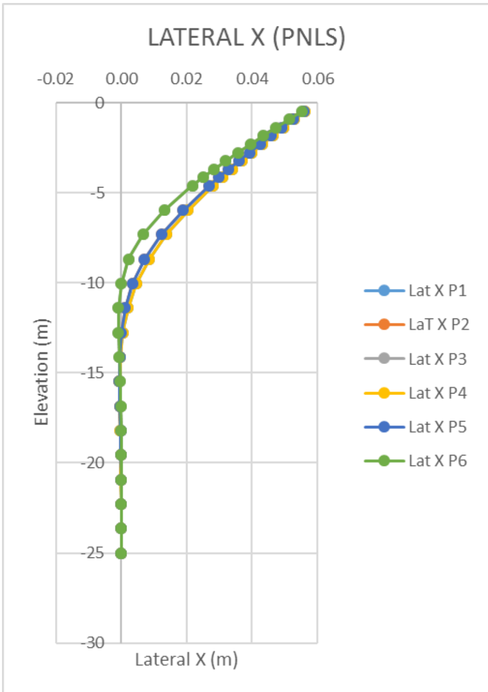


Figure 5.24.2 Non-Linear Lateral X Profile

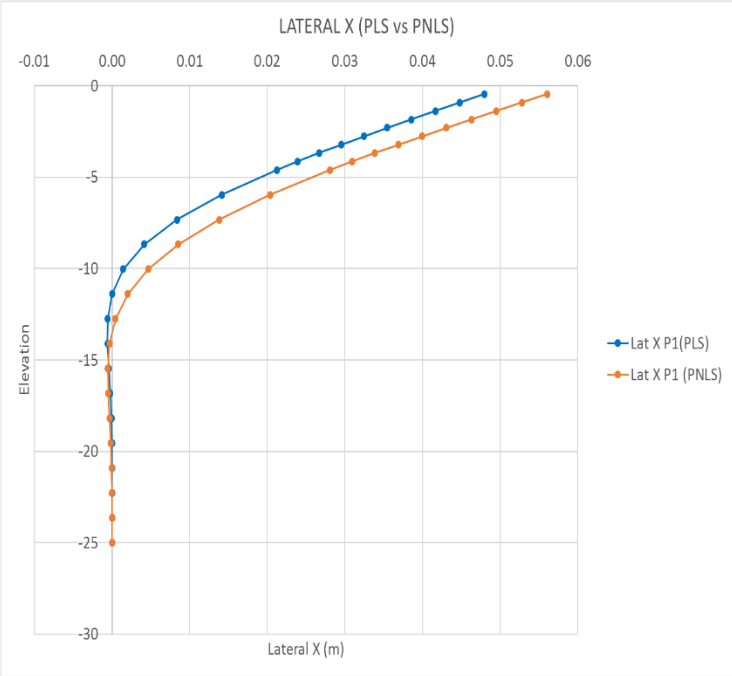


Figure 5.24.3 Linear vs Non-Linear Lateral **Displacement Profile**

2.2.1.6 Demand/Capacity Ratio

FB-MultiPier calculates the demand/capacity ratio for each cross section used in the analysis. The Demand/Capacity ratio as well as the interaction diagram are only calculated when full cross sections are specified (either linear with full cross section or nonlinear). The Demand/Capacity ratio is an estimate of the percentage of the cross sections' capacity that has been reached for that particular loading state.

The values of predicted maximum D/C Ratio considering the linear material analysis of the piles are between 30% and 40% corresponding to the static loads (Figure 5.25.1). Similarly, the values of maximum axial force predicted by the FB-MultiPier considering the non-linear material analysis of the piles are between 40% and 50% corresponding to applied lateral loads (Figure 5.25.2) . It is observed that D/C Ratio values predicted form the FB-MultiPier analyses using both linear and non-linear material behavior of pile have similar trend and shape at all applied loads, but the non-linear analysis produce about 20% larger value of D/C Ratio as shown in Figure 5.25.3.

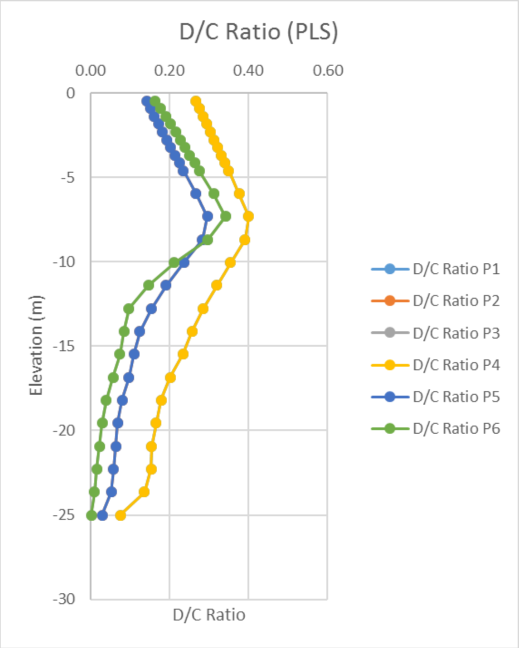


Figure 5.25.1 Linear D/C Ratio Profile

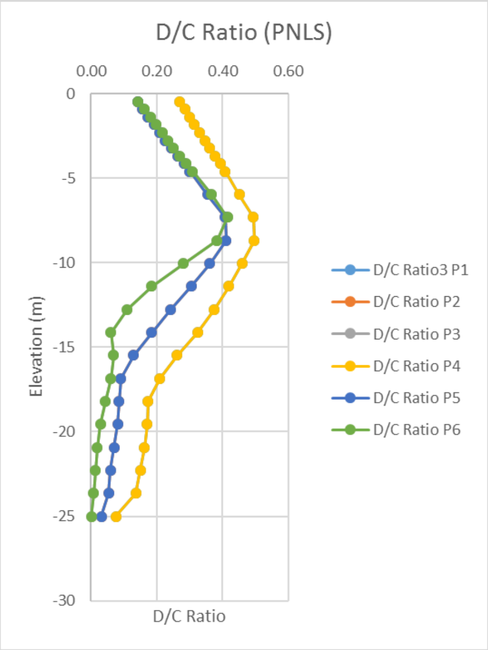


Figure 5.25.2 Non-Linear D/C Ration Profile

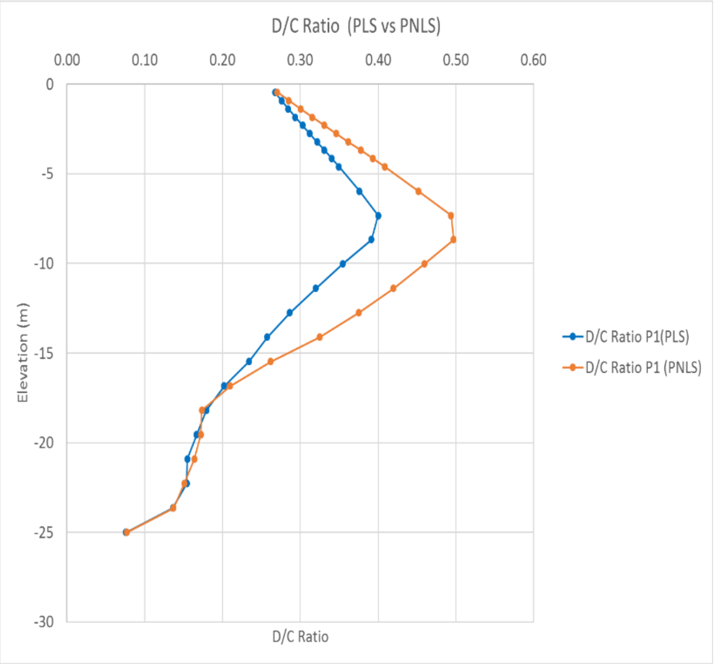


Figure 5.25.3 Linear vs Non-Linear **Demand Capacity Ratio Profile**

2.3 PIER COLUMN RESULTS

Results can be viewed after reaching a converged solution to the problem. This completes the modeling phase for the pier columns. The pier columns results is view the maximum and minimum results for the pier columns .When the analysis is completed, this will highlight the pier with the D/C ratio, show the force results only (Shear 2 and Axial forces) and the moment 3.The resulting plots are show in Figures 5.26 and 5.27.

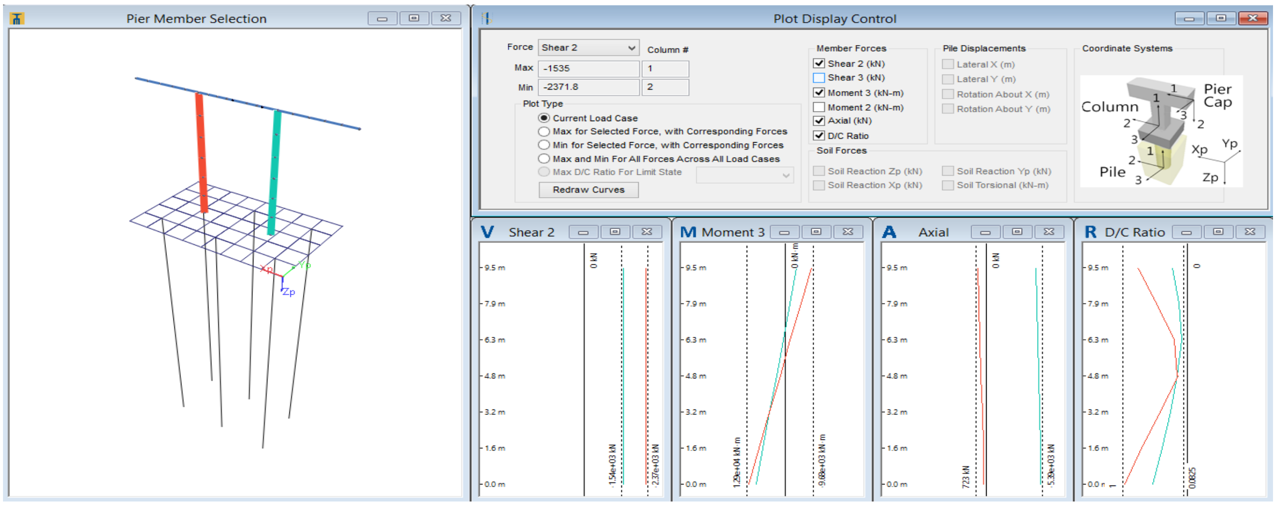


Figure 5.26 FB-Multipier Screenshot for Linear Pier Column Results

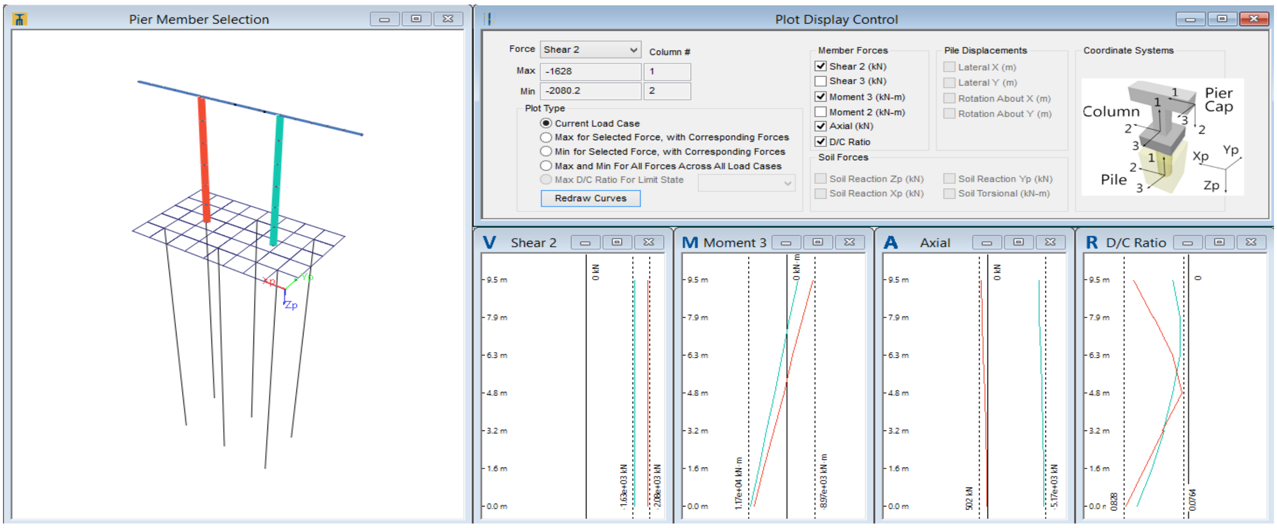


Figure 5.27 FB-Multipier Screenshot for Non-Linear Pier Column Results

2.3.1 DISCUSSION

2.3.1.1 Shear Force Profile

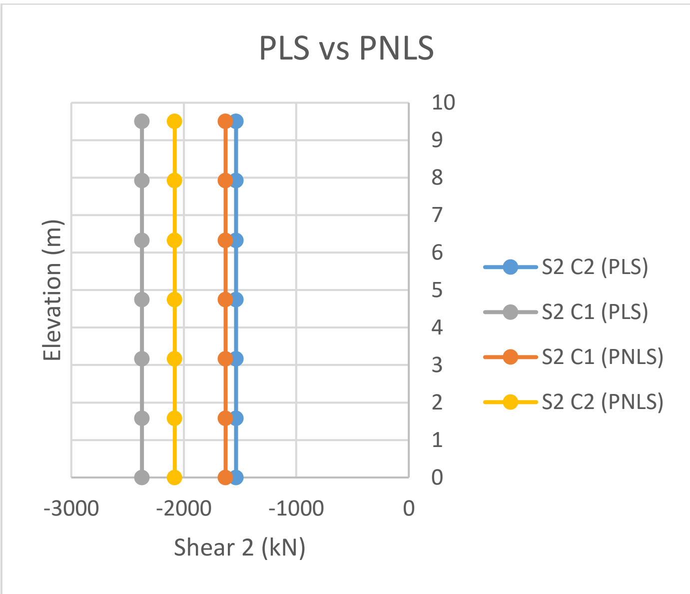


Figure 5.28 Linear vs Non-Linear
Shear Force Profile

2.3.1.2 Moment Force Profile

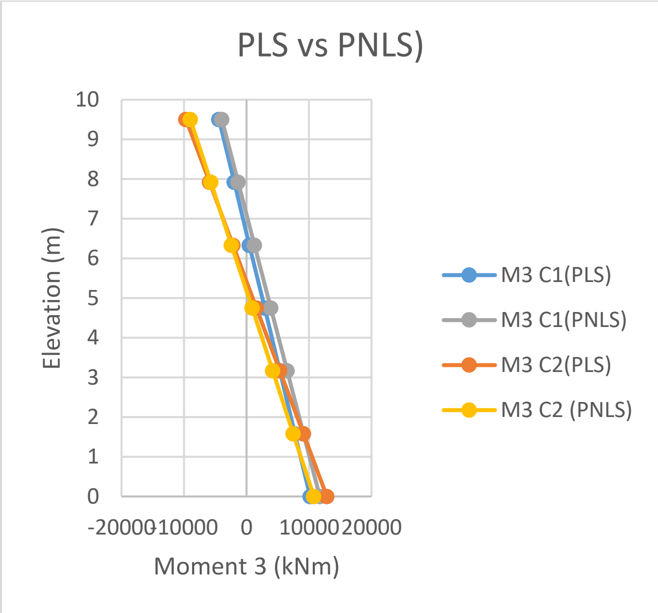


Figure 5.29 Linear vs Non-Linear
Moment Force Profile

2.3.1.3 Axial Force Profile

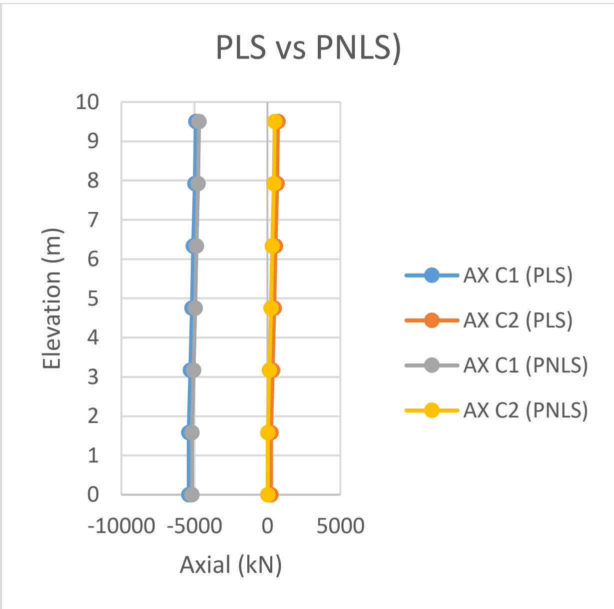


Figure 5.30 Linear vs Non-Linear
Axial Force Profile

2.3.1.4 D/C RATIO

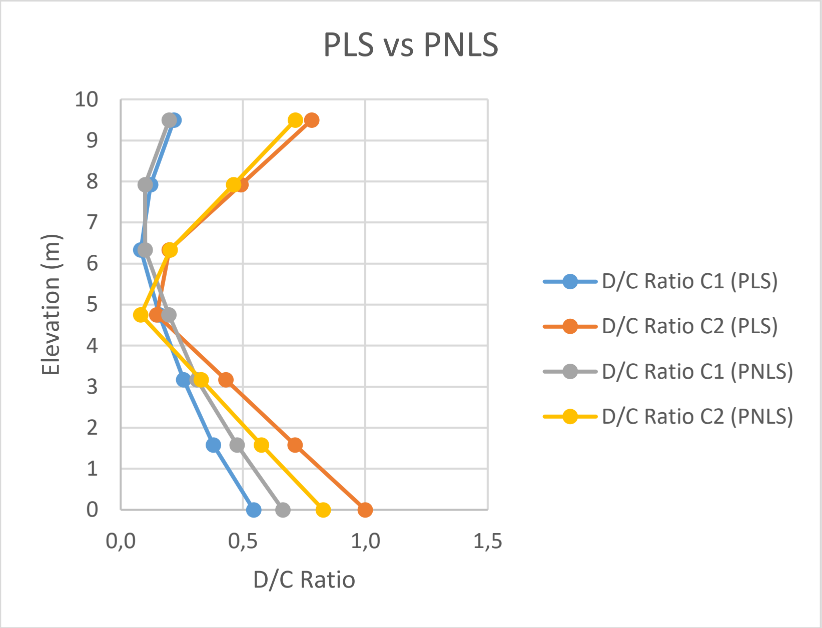


Figure 5.31 Linear vs Non-Linear
D/C Ratio Profile

Conclusion

This study highlights the significance of numerical modeling in analyzing the behavior of bridge piers founded on pile groups in cohesive soil. By conducting experiments using the FB-Multipier program and a comprehensive dataset, we were able to simulate the response of bridge piers to various acting forces. FB-MultiPier calculates the response of the bridge pier and pile group, providing output such as bending moments, shear forces, axial forces, and displacements. Based on the analysis results, optimize the design by adjusting parameters such as pile spacing, diameter, length, or shape of the pier. Iteratively analyze and refine the design until the desired performance and safety criteria are achieved using the D/C Ratio. It is observed that D/C Ratio values predicted form the FB-MultiPier analyses using both linear and non-linear material behavior of pile have similar trend and shape at all applied loads. This research contributes to the advancement of numerical modeling techniques and enhances our understanding of bridge performance in cohesive soil conditions. The findings have practical implications for optimizing bridge design and developing efficient foundation systems, ultimately leading to safer and more reliable bridge structures.

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