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1. INTRODUCTION

This document outlines the Geotechnical Engineering design parameters to be considered when designing foundations and the possible engineering problems which may be encountered during foundation design. This document also considers the stability and deformation of excavations, embankments and (earth) retaining structures.

The design of supporting infrastructure such as roads, underground piping etc., shall be done using similar considerations.

2. SUPPORTING CLAUSES

2.1 SCOPE

This document shall be used as a reference document.

2.1.1 Purpose

This document serves as a guideline for Geotechnical Engineering design of Civil Engineering Works.

2.1.2 Applicability

This document shall apply throughout Eskom Holdings Limited Divisions.

2.2 NORMATIVE/INFORMATIVE REFERENCES

Parties using this document shall apply the most recent edition of the documents listed in the following paragraphs.

2.2.1 Normative

[1] ISO 9001 Quality Management Systems

[2] Byrne, G. &, Berry, A.D., “Franki: A Guide to Practical Geotechnical Engineering in Southern Africa, 4th Edition (2008)

[3] Skandari, L.E, &, Kalantari, B. (2011). “Basic Types of Sheet Piling and Their Application in the Construction Industry – A Review”, Volume 16.

[4] California Department of Transport (2013). “CALTRANS Maintenance Blasting Manual”

2.2.2 Informative

None

2.3 DEFINITIONS

General Definitions:

Definition	Description
Shall and Should	The word 'shall' is to be understood as mandatory and the word 'should' as strongly recommended to comply with the requirements of this manual.
Principal	The party which initiates the project and ultimately pays for its design and construction. The principal will generally specify the technical requirements. The principal may also include an agent or consultant, authorized to act for the principal.
Contractor	The party which carries out all or part of the design, engineering, procurement, construction and commissioning for the project. The principal may sometimes undertake all or part of the duties of the contractor.

Technical Definitions:

Definition	Description
Foundation	The part of the structure in direct contact with, and transmitting loads to, the ground. The prepared ground base on which structures rest.
Superstructure	The total structure excluding any foundation types.
Ultimate Bearing Capacity	**Ultimate bearing capacity** is the theoretical maximum **pressure** which can be supported without failure; allowable **bearing capacity** is the **ultimate bearing capacity** divided by a factor of safety.
Design Load to the superstructure	All loads applied to the superstructure at SLS and ULS
Forces applied to Foundations	All forces from the superstructure at SLS and ULS
Founding Stratum	The founding stratum can be defined as a layer of material (natural or artificial) which can be used to support a foundation and structure
In-situ testing	Testing prescribed by the geotechnical engineer in conjunction with the structural/civil engineers which is conducted in the original position in the ground.
Consolidation	The process in which the soil volume decreases (causing settlement) due to draining of excess pore water pressure and increasing in effective soil stresses.
Liquefaction	Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading (and the addition of water).
Collapsible soil	Collapsible soils are soils with an open grain structure, which have individual grains connected by a bridge of fine material (clays/silts). An introduction of moisture and/or vibrations and/or loading results in a softening of the bridge of fine materials, thereby causing a collapse of the grain structures.
Expansive soil	Expensive soils are soils which expand in volume with the addition of water (heave) or decrease in volume when drying (subsidence).
Stemming	Inert materials used to confine the gasses generated during blast detonation

2.3.1 Disclosure Classification

Controlled Disclosure: Controlled Disclosure to external parties (either enforced by law, or discretionary).

2.4 ABBREVIATIONS

Abbreviation	Description
CALTRANS	California Department of Transport
CFA	Continuous Flight Auger
CoE	Centre of Excellence
DTH	down the hole hammer
ISO	International Organization for Standardization
kPa	kilopascals
PPV	Peak Particle Velocity

Abbreviation	Description
SI units	International System of Units
SLS	Serviceability Limit State
ULS	Ultimate Limit State

2.5 ROLES AND RESPONSIBILITIES

Not applicable

2.6 PROCESS FOR MONITORING

None

2.7 RELATED/SUPPORTING DOCUMENTS

[5] 240-57127951: Standard for the Execution of Site Investigations

[6] 240-57127953: Execution Of Site Preparation And Earthworks Standard

3. GEOTECHNICAL AND FOUNDATION ENGINEERING STANDARD

3.1 BASIC APPROACH TO GEOTECHNICAL ENGINEERING

3.1.1 Geotechnical Terms and Definitions

The design of foundations ensures the long term stability of a structure by taking into consideration the soil bearing capacity, the settlements and the load characteristics of the structure i.e. static or dynamic.

Variability in the ground profile will result in variable soil geotechnical properties from one location to another. The geotechnical properties of the soil should be assessed and advised by an experienced and professionally registered engineering geologist/geotechnical engineer.

3.1.2 Site Investigation

Should there be a lack of existing geotechnical information or in instances where the existing geotechnical information is deemed insufficient; a site investigation shall be conducted to determine the characteristics and variability of the subsurface geology and geohydrology. The geotechnical testing regime will also make considerations for other factors interacting with the proposed work or structure. (Refer to 240-57127951: Standard for the Execution of Site Investigations).

A geotechnical and chemical analysis of soil and ground water shall be carried out to establish whether the soil and/or groundwater conditions may affect the proposed foundation or drainage system.

Attention must be given to possible changes in the soil and groundwater conditions (qualitatively and quantitatively) caused by the engineering works, both during construction and when in service.

3.1.3 Soil and Rock Classification

Soils and rocks should be classified in accordance with latest acceptable local and international practices. (Reference 240-57127953: Execution of Site Preparation and Earthworks Standard)

3.1.4 Loads on Foundations

3.1.4.1 General

The loads on the foundations shall be calculated according to the latest applicable National/ International Standards.

3.1.4.2 Static loads

Static loading is defined as any load on a structure that does not change in magnitude and position over time. Static loads comprise of dead loads and live loads. Dead loads are defined as static forces which remain relatively constant over time, and can exist in tension or compression. Live loads are defined as unstable or moving loads. Live loads also include temporary or transient loads acting on a structure e.g. people, vehicles.

3.1.4.3 Dynamic loads

Dynamic loads are non-constant loads acting on a structure e.g. wind, reciprocating machinery. Dynamic loads may be defined as imposed forces which are in motion. Dynamic loads exert varying forces on a structure.

3.1.5 Seismic Activity

The area in which the proposed works or structures are to be located shall be investigated to determine the occurrence of natural or induced seismic activity. Existing applicable national or local seismic codes shall be adhered to.

In any seismic prone area a special site investigation shall be carried out to analyse the possible ground movements, and collapse potential.

For areas displaying saturated to over saturated ground conditions and experiencing large amounts of induced agitation (for SA only), the liquefaction potential of the soil will need to be assessed.

3.1.5.1 Induced Seismic Activity (Blasting) (*CALTRANS Maintenance Blasting Manual, 2013*)

Blasting is the process of physically breaking down solid rock; or cemented material with hardness's equal to that of rock; by means of explosives. The decision on whether or not blasting is to be implemented will depend on the type and confinement of the structure and the prevalent geological condition. The sections below are intended for the reader to understand the concept of and considerations when blasting. It is important to note that the blast design must be done by a blast specialist.

Types of Explosives:

Explosive Type	Description
Dynamite	Cap sensitive explosive, usually sensitized with nitro-glycerine
Water Gels, Slurry, Emulsions	Explained as a single type of explosive for this explanation. These types of explosives may or may not be cap-sensitive, depending on formulation. NB. Cap-Sensitive are classified as high explosives. These explosives are mixtures of oxidizers, fuels and sensitizers, usually containing between 5% to 45% water.
Blasting Agents	Defined as any material or mixture consisting of a fuel and oxidizer, in which one of the ingredients classifies as an explosive. E.g. ANFO which is 94% prilled ammonium nitrate and 6% diesel fuel.
Binary (two component) Explosives	Consist of two separately packaged material, neither of which is an explosive. When combined the two materials become an explosive compound.

3.1.5.1.1 Properties of Explosives:

- Detonation velocity is the rate at which the detonation wave progresses through the explosive. This parameter describes the explosive behaviour. A high-velocity indicates a shattering effect whereas a lower velocity with good gas build-up results in a heaving action (not to be confused with soil heave).
- The density or specific gravity of an explosive is measured in grams per cubic centimetre. The loading density is calculated for the specific gravity of the explosive and the diameter of the explosive column.
- The strength of an explosion is a function of the velocity, specific gravity and the gas generated. The Strength of the explosive relates to the amount of work an explosive does.
- Water resistance is the ability of an explosive to withstand exposure to water and still perform satisfactorily. The water resistance is rated as either a rating from Poor to Excellent, or assigned a number from 1 [good] to 4 [poor].
- The Fume Class of an explosive is determined by the quantity of toxic gas produced during detonation. Rated as 1 [least amount of noxious gas] to 3 [highest]. Fume Class 1 is used unless there is adequate ventilation.
- Detonation pressure is the pressure of the detonation wave as it progresses through the explosive. The detonation pressure is a function of the velocity, density and ingredients within the explosives.
- Powder Factor is the total explosives in the blast/volume of rock blasted expressed as kg/ton or m^3

The various methods of blast initiations have not been included in this document.

3.1.5.1.2 Blast Design

It is important to note at the onset that Blast design is not an exact science. Successful blasters often use rules of thumb to design their blasts and require a significant amount of experience to execute successful blasts.

Various factors are to be considered for blast design; the outline below is intended as guidelines for blast design but rather a review of variables considered during successful blasting operations. The final responsibility for the blast design must always be assumed by the blaster.

- Shot Planning – consideration is made to the following:
  - Breakage desired: size of excavation /handling equipment, size of crushing equipment (where applicable)
  - Rock Quality and Characteristics: hardness, moisture, variability of profile, jointing/bedding
  - Site Limitations: Structures/property in close proximity to blast site, utilities which may be damaged, traffic
  - Safety Limitations: protection from fly-rock, weather considerations (humidity/lightning), electrical hazards, ventilation, rock fall hazard impact
  - Equipment and Materials Limitations: drilling equipment, explosive type and quantity, accessories (blast machine capacity, firing line length)
- Blast Calculations – when considering blast calculations, reference is often made to powder factors (expressed as kilograms of explosive per cubic meter of rock). There are various methods for blast calculations however the powder factor is used as a yardstick used by blasters to compare similar blasts.

- Boulder Blasting: involves the blasting of rock which is less confined, therefore the probability of fly-rock is increased.
  - Block Holing – required that explosives be loaded into holes drilled into rock, typically drilled 55% to 60% of the into the depth of the rock
  - Mud Capping (Cone Blasting) – A practice used to break up large boulders or rock in a work area. A capped piece of explosive is laid on the boulder and is "capped" with either mud or wet muck to contain the force of explosion. Used when drilling into rock is not possible or practical. This method must be used in areas where air blasts are possible. A high velocity explosive with good shattering capability is used.
  - Snake Holing - used for blasting partially buried boulders. A hole is punched under the rock and is loaded with the required explosives.
  - Seam Blasting - loading of explosives directly into deep seams or cracks in the rock. Confinement of the explosive occurs by the use of stemming materials or mud-capping.
  - Production Blasting – Large number of holes are drilled and loaded (as seen in quarrying, through-cuts)

- Presplit Blasting - Unstemmed holes which take into account the spacing, burden, uncharged length and powder factor. All holes are fired on the same day, or in groups of 5 or more.
- Smooth Blasting - Stemmed holes which take into account the spacing and burden. As many holes as possible are fired on one day.

3.1.5.1.3 Air Blasts:

- Air Blasts are an impulsive sound or airborne shock wave generated by an explosive blast and the resulting rock fragmentation and movement, sources include: Detonations at surface
- Rock Displacement, either at face or around the collar of the borehole
- Gas escaping through fractures in rock/blown out stemming
- Variables affecting the character and level of the air-blast:
  - Charge weight
  - Distance
  - Delay intervals between holes
  - Face orientation
  - Explosive confinement
  - Weather
- Good blast practise involving stemming, hole patterns, powder factor and timing will help reduce excessive air-blast.

3.1.5.1.4 Vibration

An undesirable effect of blasting, Vibration is usually described as the displacement, acceleration or velocity of a particular point in the ground. As pressure waves generate in the surrounding rock, the pressure waves move from the blast holes causing seismic waves which displace particles. The particle movement is used to measure the magnitude of the blast vibration. Vibration may be expressed as Peak Particle Velocity (PPV).

Factors which contribute to blast vibration include:

- Excessive number of holes firing/rows in a blast
- Excessive overburden on the hole
- Incorrect timing patterns
- Underloaded holes

3.1.6 Design Computations and Report

A design report shall be prepared containing the following information as a minimum:

- The results of site investigations used for the design
- Geotechnical recommendations
- Brief outline of the design methodology
- All calculation results
- All factual and interpretive results shall be clearly distinguished

All geotechnical computations shall be made using SI units. The calculation methods are not prescribed by this manual.

Calculation methodology shall be based on the latest applicable National/ International Standards.

3.1.7 Effects on Adjacent Structures

Implementation of a new structure in the vicinity of an existing structure may have negative effects on the existing structure/s and/or on the new structure. In the design and during construction all possible interactions shall be analysed.

The design engineer shall ensure that all necessary considerations have been made when assessing; the construction impact on adjacent structures and the impact to the intended structure.

During Construction, the Contractor should conduct checks which shall include but not be limited to:

- Excavations or fills which may cause horizontal and vertical soil deformations that may affect adjacent piles or shallow foundations.
- Dewatering which may cause settlements of adjacent structure or cause down drag loads on piled foundations and it shall be investigated prior to carrying out any dewatering.
- Loading of a new foundation adjacent to an existing foundation which may cause tilting of either one or both foundations.
- Vibrations due to pile driving that may cause settlements of adjacent structures or damage to sensitive equipment in adjacent structures.

Where risk of damage to adjacent structures exists, the principal shall be consulted before the implementation is started.

3.2 SPREAD FOUNDATIONS ON SOIL

3.2.1 Foundation Depth

Foundation depths shall be selected with reference to the following requirements:

- An adequate bearing stratum shall be reached under the total area of the foundation, and

- The foundation level shall be below the zone affected by erosion and other influences (seasonal shrinkage and swelling, tree roots, etc.).

Foundation depths shall be designed based on the advisement from the geotechnical investigation report.

3.2.2 Ultimate Bearing Capacity

The ultimate bearing capacity shall be calculated taking into account the distribution of stresses in the subsoil.

Ultimate Bearing Capacity shall be calculated using the latest applicable National/International Standards.

Allowable bearing capacity can be expressed at the Ultimate bearing capacity divided by the allowable factor of safety. The Allowable bearing capacity shall comply with all relevant National and International Codes and Standards.

3.2.3 Settlement Behaviour

The settlement behaviour of the foundation shall be calculated from the stress distribution in the subsoil, and be based on compression parameters of the substrata derived from laboratory tests within the geotechnical report.

Both the total and differential settlement as well as the increase of the settlements with time shall be calculated, calculations shall be based on the latest applicable National/International Standards. See Section 3.6.3.

Where relevant, effects of vibrations on long-term settlement (tilting) shall be taken into account.

3.2.4 Allowable Load

In general, the allowable load on a foundation is determined by the (differential and absolute) settlements that can be accepted. The settlements shall be calculated as a function of the loading and shall be compared with the absolute and differential settlements that can be tolerated by the superstructure. Calculations shall be based on the latest applicable National/International Standards.

3.3 PILE FOUNDATIONS IN SOIL

3.3.1 General

In general, the design of pile foundations is a specialist activity and shall be designed by the contractor's pile designer. The type of piling system that will be implemented shall be based on the recommendation from the geotechnical report, and other structural factors (mentioned briefly below).

Pile foundations are used for ground profiles in which:

a. The soil extends beyond 3m depth with the surface soils being incapable of supporting the load of the structure.

b. The soil profile may contain soils with variable properties i.e. high compressibility soils at depth etc.

c. Property lines may be narrow

The main function of piled foundations is to transfer the load on a foundation to deeper (soil) strata which are capable of carrying the working load in such a way that:

- An adequate safety factor against loss of stability is provided, and

- The foundation settlement/deformation is less than the absolute and differential settlements/deformations that are allowable for the superstructure and the equipment supported.

The design and testing of piles shall take into account both short-term and long-term behaviour of the pile foundation.

Bearing Piles – a pile driven or formed in the ground for transmitting the weight of a structure to a competent bearing stratum to carry the full load at the toe of the pile. In some instances resistance developed by friction along the surface can be used.

Friction Piles - piling which receives its principal vertical support from the skin friction between the surface of the buried pile and the surrounding soil. Note; friction piles cannot be used in areas with expansive soils.

3.3.2 Type of Soil Investigation

Refer to:

- 240-57127951: Standard for the Execution of Site Investigations
- 240-57127953: Execution of Site Preparation and Earthworks Standard

3.3.3 Types of Piles (*A Guide to Practical Geotechnical Engineering in Southern Africa, 4th Edition, 2008*)

The type of pile to be implemented will be based on the geotechnical recommendation as seen in the geotechnical report. This recommendation will be coupled by a site investigation report. The type and design of the piling system is a specialist activity and shall be done by the pile designer.

The following is general information on different piling systems:

Types of Piling Systems:

a. Franki driven cast in-situ piles
b. Driven tube piles
c. Precast piles
d. Steel H piles
e. Timber piles
f. Auger piles
g. Underslurry piles
h. Continuous flight auger (CFA) piles
i. Forum bored piles
j. Oscillator piles
k. Caisson piles

Table 1: Summary of Piling Systems (after Byrne, G. & Berry, A.D. (2008) pp. 68)

6.0 SUMMARY DETAILS OF PILING SYSTEMS

Ref.	Pile Type	Nom. Shaft Diameter (mm)	Typical Working Load (kN)	Max. Tension Load (kN)	Max. Rake	Max. Depth (metres)	Cost per kN per Metre
	DRIVEN CAST-IN-SITU						
7.1	**Franki Pile**						
	Mini	250 or 300	250 - 450	75	1:4	8	Med/High
	Light	355	500 - 650	150	1:4	15	Medium
	Medium	410	700 - 900	250	1:4	18	Medium
	Heavy	520	1000 - 1500	350	1:4	18	Med/Low
	Super Heavy	610	1600 - 2000	450	1:4	18	Med/Low
7.2	Steel Tube Pile	300 - 1200	Up to 10 MPa on shaft	Determined by friction	1:4	20 - 50	High
	DRIVEN PRE-FORMED						
7.3	Precast Pile	250, 300 and 350 sq	750 - 2000	Determined by friction	1:4	Unlimited	Low
7.4	Steel H-Pile	Different Steel H sections	Maximum Stress 165 MPa	Determined by friction	1:4	Unlimited	High
	BORED CAST-IN-SITU						
7.5	Auger Pile	300 - 2000	3 - 8 MPa on shaft	Determined by friction	1:4	50	Low
7.6	Underslurry Pile	600 - 1500	Up to 8 MPa on shaft	Determined by friction	Vert. only	50	Medium / High
7.7	CFA Pile	300 - 750	Up to 6 MPa on shaft	Determined by friction	1:10	25	Low
7.8	Full Displacement Screwpile	300 - 700	400 - 3000	Determined by friction	1:6	25	Low
7.9	Forum Bored Pile	410	500 - 750	200	1:6	12	High
7.10	Oscillator Pile	900 - 1500	Up to 10 MPa on shaft	Determined by friction	1:4	60	High
7.11	Rotapile	255 - 610	300 - 2500	Determined by friction	1:8	40	High
7.12	Micropile	Up to 300	Up to 1000	Determined by friction	1:2	30	High

3.3.3.1 Franki Driven Cast in-situ Piles

Franki piles are suited for structures varying from single storey residential to multi storey office building construction. This is due to the variety of pile sizes and advantage of enlarged bases. The Franki pile has excellent load/deflection performance, and tension load capacity.

Considerations to be made include:

- Vibrations associated with the driving of the pile tube and the formation of the enlarged base
- Pile heave in saturated cohesive soils

It is important to note that the main feature of the franki pile is the enlarged base at the toe of the pile, which increases the end bearing area.

3.3.3.1.1 Conditioning of the soil

Conditioning in sandy soil profiles in achieved by the act of driving the piling tube into the ground. As more piles are driven, the soil strength increases.

3.3.3.1.2 Limitations

- **Limited depth**

  One of the limitations to the Franki pile is the limited depth to which the standard pile can be installed i.e. 8m for mini – 18m for Medium/Heavy/Super Heavy. Therefore, if the water table is not high, the depths will need to be increased using extension tubes.

- **Pile Heave**

  Pile heave can be defined as the phenomenon in which a previously installed pile is lifted by an upward movement of the surrounding soil. This is caused by the driving of an adjacent pile. Pile heave commonly occurs in saturated clayey or silty soils, and occurs with driven displacement type piles and not bored piles.

  Pre-drilling or coring piles using specialist vibration installation techniques may reduce the amount of heave as displacements occur over reduced depths.

3.3.3.2 Driven Tube Piles

Make use of high cost steel tubes, small pile sizes are commonly used for underpinning of houses and light buildings with poor access and/or limited headroom. Medium pile sizes may be used for piling new column foundations within existing buildings or in difficult access areas. Larger sizes may be used for river bridge foundations and in marine construction.

Driven tube piles achieve considerable depths (>60m in a suitable profile). Due to the ability of driven tube piles to be permanently cased, they are well suited for river/marine environments.

The steel piling tubes can either be open or closed ended. For closed ended tubes, the toe end of the piling tube is sealed off with a steel plate therefore there is a full displacement during driving. Open ended steel tubes are associated with temporary staging structures as well as marine work.

3.3.3.2.1 Limitations

- **Split pile tubes**

  If the correct driving processes and appropriate casing thicknesses are adhered to, splitting should not occur. It is the responsibility of the contractor to ensure that splitting does not occur.

- **Pile Heave**

  As these piles are displacement type, pile heaves may occur in saturated cohesive soil profiles.

3.3.3.3 Precast Piles

Due to the noise levels associated with the driving of the precast piles, they are seldom adopted in heavily populated areas. Precast piles are economical for deeper soil profiles, precast shafts are often deemed as higher quality than that of in-situ shafts.

3.3.3.3.1 Rock Shoes

Where precast piles are required to penetrate hard rock/cobble/boulder layers and/or used in sloping rock faces; they are normally fitted with rock shoes. Rock shoes may be end plates to sophisticated hardened steel points depending on the requirement.

3.3.3.3.2 Limitations

Precast piles being displacement piles can experience pile heave.

3.3.3.4 Steel H-piles

Steel H piles are not commonly used as they are often expensive. Given the full steel frameworked nature, the pile has good penetrating ability. These piles can be installed to considerable depths and are ideal as soldier piles. The end-bearing ability of Steel H piles is limited unless driven into rock. Steel H-piles must be designed with careful consideration for corrosion protection.

3.3.3.4.1 Limitations

- **Pile Heave**

  Pile heave risk is low as there is limited displacement when driving steel H-piles.

- **Bent Pile shafts**

Deflection or bending may occur during the driving process, particularly when there are obstructions in the ground profile. A pile load test will need to be implemented to indicate if the performance of pile is acceptable. It is the responsibility of the contractor to ensure that piles are driven adequately.

3.3.3.5 Auger Piles

Ideally suited to partially saturated, cohesive and residual soils. Auger piles are widely implemented with Eskom power stations. Auger piles are suited to large and small structures and are available in a wide range of pile sizes. Auger piles often provide an economical solution to heaving soil profiles. Auger piles can be installed to depths > 50m using hydraulic piling rigs, and can accommodate boulder layers with some limitations.

3.3.3.5.1 Pile Heave

When soil heaves in a piled area, there is a friction component transfer between the soil and the pile. This friction imparts an uplift force on the pile shaft. If this force exceeds the axial compression load on the pile, then the pile shaft will be in tension. If the tension force is not high, it can be resisted by the shaft reinforcement and the associated anchorage provided by the length of the pile below the “active” soil zone.

In circumstances where the tension forces are potentially high, shaft reinforcement may be developed to reduce the friction transfer.

3.3.3.5.2 Limitations

- **Collapse of sidewalls**

  Auger piles however, have a considerable risk of excavation side wall collapse. This may be remediated with temporary casings but this is an expensive solution.

- **Water Ingress**

  Below the water table, there is considerable risk of excessive water ingress. This is particularly evident if soft rock occurs at the toe of the pile. This may be remediated by analysis of the rate of inflow, and implementation of a pump. The pumping must be coupled with concreting of the shaft.

  If the rate of inflow is too fast, the pile will have to be cast under water, this will however require temporary casing. Bentonite may be used at the head of the pile during the drilling process; however; this is an increased cost.

- **Boulders**

  The presence of boulders may be problematic for factors such as smaller diameter piles, hardness and concentration of boulders, type of soil matrix and presence of water. Under certain circumstances, piling through boulders may be achieved albeit at a lower production rate. It is for this reason that a geotechnical report must advise the implementation of Auger piles.

3.3.3.6 Underslurry Piles

These piles are commonly used for heavily loaded structures in fully or partially saturated soil profiles which may result in unstable side wall conditions for augered piles. “Underslurry” piles refer to piles excavations whereby the pile is excavated under a head of bentonite slurry (bentonite + water). This bentonite slurry prevents the collapse of the pile excavation.

Underslurry piles are economical for heavy structures on saturated and unstable soil profiles, with a range of pile sizes from 600mm-1500mm in diameter. Underslurry piles can be installed to a depth of > 50m.

3.3.3.6.1 Limitations

Underslurry piles have a high establishment cost. These piles usually require a large site area and are exclusively suited to heavy structures. The bentonite slurry contaminated spoil must be disposed of in an approved waste site.

- **Collapse of the pile shaft excavation**

  There is a risk of side wall collapse; however this can be reduced by adherence to correct procedures and slurry specifications.

- **Delays before concreting**

  If there are unforeseen delays in concreting of the pile, this will result in an increase in thickness of the bentonite cake thereby reducing the pile friction capacity.

3.3.3.7 Continuous Flight Auger (CFA) Piles

The CFA piles are fast economical piling systems however only in suitable soil profiles. In sandy-soils below the water table there is a reduction in soil strength in the vicinity of the pile due to the drilling operations. This thereby causes the load/deflection performance of the CFA pile to be inferior to the Driven pile.

Soil falling off the flight can contaminate the concrete/grout after the pile has been cast. CFA piles have to be cast to ground level; hence there is a waste of concrete and excess trimming. CFA piles also make use of a bentonite slurry to prevent excavation collapse.

3.3.3.7.1 Limitations

- **Necked pile shafts**

  This is caused by incorrect extraction of the flight, therefore monitoring of the flow rate of concrete/grout and extraction rate is required. This is the responsibility of the contractor.

- **Reduction in pile capacity**

  The drilling of the auger into the ground results in a decompression of the soil profile. The CFA end-bearing component; when founded on sandy soils below the water table; is significantly affected by the drilling operations. It is for this reason that such soil profile situations are "ignored" and the pile is designed as a friction pile. CFA piles founded on stiff cohesive soils and rocks have a full end-bearing component, and shall be designed as such.

- **Boulders**

  Boulders larger than 1/3 the pile diameter may present a problem for auger excavations. In these situations, the only viable solution may be to relocate the pile.

3.3.3.8 Full Displacement Screwpiles

These piles are "recently" introduced to South Africa owing to the established track record in other parts of the world. Economical, high production rates can be accomplished in suitable soil profiles. No soil removal is required with the soil displacement technology. However, the penetration of boulder layers/ obstructions may prevent the use of this type of piling system.

Full Displacement Screwpiles require excessive drilling torque to penetrate thicker, denser sands or stiff clays; this will need to be carefully assessed for heavily loaded foundations.

3.3.3.8.1 Limitations

The Full Displacement Screwpile is a refinement of the CFA pile and therefore experiences similar installation problems. There are difficulty penetrating boulders or thick stiff/dense soil profiles. Due to the soil displacement technology of the piling system the penetration depths are not easily achieved. Therefore, the need for enhanced drilling torque.

3.3.3.9 Forum Bored Piles

Forum Bored piles are characterised by small and light piling equipment. The system is suited to low headroom and areas with limited site access. The Forum bored pile has an enlarged base similar to the Franki pile and has a good tension capacity. However, the forum bored pile has low production rates and the system is not suited for cohesionless soils below the water table. The pile is only available in one size, and the depth is limited to approximately 12m.

3.3.3.9.1 Limitations

Due to differential head between the water table on the outside and inside of the pile tube, the soil and water are said to "boil" into the pile excavation. Boiling can only take place where a water table is present.

This "boiling" action during pile excavation may result in ground settlement around the pile vicinity. In sandy soil profiles below the water table, it is important that there is a cohesive layer immediately above the founding level into which the tube is sealed. This allows for the pile to be cleaned successfully, and the base formed.

3.3.3.10 Oscillator piles

Oscillator piles penetrate through rock and boulder layers, these piles are therefore ideal for large river bridges and for marine construction. The pile diameters are relatively large and pile sizes are limited. Therefore the Oscillator piling system is only economical for heavily loaded structures. Oscillator piles may reach depths of up to 60m.

3.3.3.10.1 Limitations

Oscillator piles are expensive and large working platforms are required.

- **Stuck Casing**

  Risk of occurring in deep piling systems [especially in 1500mm diameter]

- **Stuck Chisels**

  Chisels wedged in the casing or the sockets may take a few days to dislodge. In some instances, piles may have to be abandoned.

3.3.3.11 Rotapiles

Rotapiles effectively penetrate boulders and rocks, and socket into hard rock using the “down the hole hammer (DTH)” percussion drilling technique. This pile is useful in Karst conditions in Dolomitic land. Rotapiles may also be implemented in river bridge foundations subject to scour. The pile casings can be permanent and incorporated into the pile design to ensure pile shaft integrity in cavity formations or moving groundwater conditions. Pile sizes range from 255 mm – 610 mm diameter with diameters of up to 1000mm, and piling depths of up to 30m.

3.3.3.11.1 Limitations

Rotapiles are expensive in comparison to driven piles with equivalent load capacity. The DTH system introduces large quantities of air, which when entered into a voided subsoil may cause problems in adjacent structures.

3.3.3.12 Micropiles

Micropiles are commonly used for underpinning existing foundations, soil reinforcement in slope stabilisation and protection of buried structures. Micropiles are used to support light structures with high uplift requirements, i.e. powerlines. Micropiles may also be implemented in Dolomitic areas.

Micropile diameter is < 300mm, and is a small diameter, drilled and grouted non-displacement pile. Micropiles are heavily reinforced and carry most of its loading on the high capacity steel reinforcement.

Micropiles can be installed in sites with limited access and smaller headroom’s. Micropiles can be installed through existing foundations (for use in underpinning).

3.3.3.12.1 Limitations

Micropiles have a relatively high cost, a careful assessment of groundwater conditions must be done to ensure that the main load bearing steel reinforcement is corrosion durable. Due to the slender nature of the Micropile, buckling effects must be considered.

3.4 PILE LOAD AND INTEGRITY TESTING

3.4.1 Pile Load Testing

The load testing of piles are conducted 28 days after casting the pile foundations. The most common form of pile load testing is the static compression test. The static compression test gradually applies a load to the head of the pile while the deflection of the pile head is monitored. Static tests can be conducted laterally by applying a lateral force and in tension by applying an upward force.

Piles can be tested using dynamic or semi-dynamic loads however these are not common to South Africa.

The objectives of load tests are to:

- Verify the pile load/deflection performance meets the contract specification
- Verify the pile design parameters
- Establish the pile load/deflection characteristics – this will allow the finalisation of the foundation design
- Assess the piles ultimate capacity
- Study the pile/soil interaction
- Check the pile shaft structural integrity

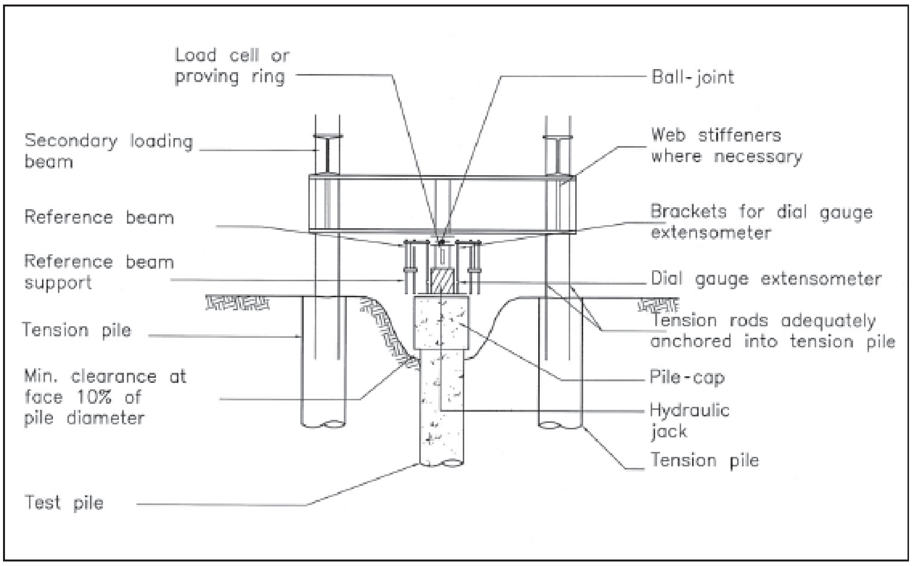

Figure 1: Typical Compression Test Arrangement (after Byrne, G. & Berry, A.D. (2008) pp 158)

The movement of the pile head is monitored using two deflectometers (minimum). The deflectometers are supported on a reference beam which is mounted on posts driven into the ground at least 2m from the test pile. The reference beam may move due to temperature effects, therefore dial gauges are sometimes used to monitor pile head movement. Most contractors will use a back-up system for monitoring pile head movement provided normally by a precise dumpy level and scale rules fixed to the pile head. In some instances it is sufficient to obtain the load on a pile from the product of the hydraulic pressure in the jack and the area of the bore of the jack.

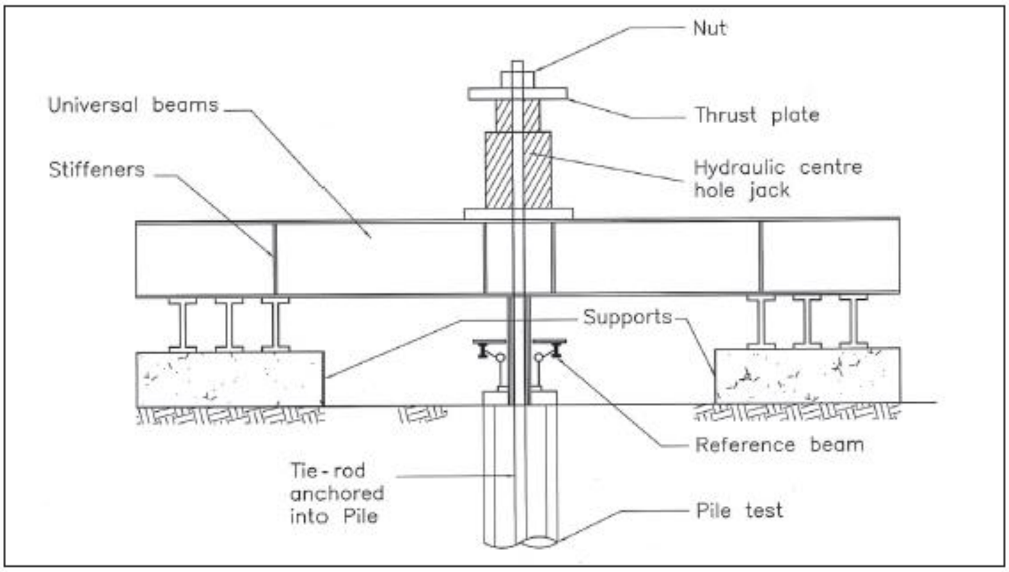

Figure 2: Tension Configuration for Load Tests (after Byrne, G. & Berry, A.D. (2008) pp 158)

Figure 2 depicts a central threaded, high capacity rod which is cast into the pile or fixed into the reinforcement. The central thread passes through a centre-hole jack with a nut providing the seat for the jack ram. The jack rests on a beam which transfers the load into two spread footings on either side of the pile.

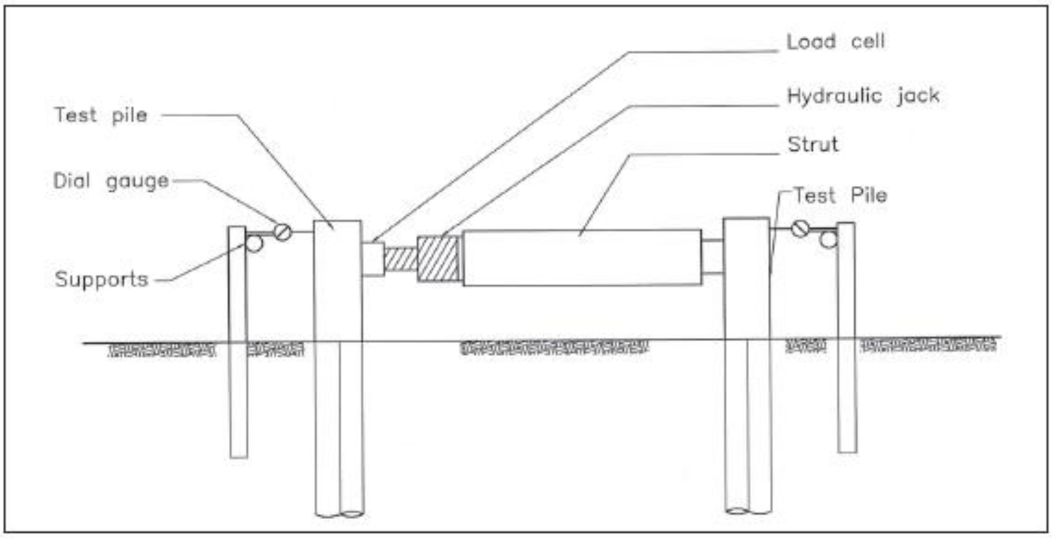

Figure 3: Lateral Configuration for Load Tests (after Byrne, G. & Berry, A.D. (2008) pp 158)

Figure 3 depicts a lateral load test. The test is easily conducted by jacking two test piles apart applied through the centreline of the piles (eliminating torsional effects). Measurement of the lateral movement is recorded by two deflectometers (minimum) mounted on a reference beam with supports clear of the zone of influence.

3.4.2 Pile Integrity Testing

Pile Integrity tests are non-destructive test methods to aid in the detection of defects in the structural integrity of pile shafts. These types of tests are not definitive and are subject to interpretation. Any defects indicated by integrity testing should be investigated by positive testing methods.

Integrity tests are conducted using either sonic or nuclear technology. Sonic methods involve the collection of data from reflections of sonic waves generated by tapping the head of the pile (Sonic Impact), or by the collection of data by a collector of sonic waves generated by an emitter, both the emitter and collector being lowered down the pile in separate small diameter tubes cast into the pile shaft (Sonic Logging). Nuclear testing adopted a similar approach to Sonic Logging however a nuclear isotope is used to generate the signal.

3.5 FOUNDATIONS ON ROCK OR CEMENTED SOILS

3.5.1 General

For foundations in weak, heavily weathered rock or cemented soil two design analyses shall be made:

- One assuming that the weathered rock mass/cemented soil behaves as a soil, see Section 3.3, and
- The other assuming rock

It is important to note the degree of weathering, in some instances weathered rock will still retain the strength of the rock. This however may deteriorate as the weathered mass is exposed to weathering elements, i.e. groundwater, sun exposure, etc.

With regards to the geotechnical logging of the weathered material, rock masses that have been weathered to a state in which more than 20 fractures occur per 10cm; this will be classified as a soil.

By thorough site investigation and geological study, it shall be assured that no unknown discontinuities [occurring at relatively shallow depth] occur in the rock below the foundation. No foundation shall be constructed on 'live' fractures or major fractures, i.e. Faults.

Possible effects of natural or induced seismic activity shall be analysed.

In the design, the top level of the bearing rock shall be defined. The bearing strength of the rock stratum shall be identified by in-situ testing and laboratory testing [reference site investigation document]

The geotechnical investigation report shall ensure that no structure is founded partly on sound rock and partly on soil or weak/highly weathered rock.

When rock fill is applied underneath the foundation, the requirements stated in the procedure for Site preparations and earthworks shall be fulfilled. [Reference 240-57127953: Execution of Site Preparation and Earthworks Standard.]

For backfill implementation partially under foundations, in which the foundation straddles the boundary of natural ground and implemented fill materials, the Contractor shall ensure that the backfill is adequately placed [under an advised backfill specification] to give the desired kilopascals (kPa) yield equal/closely similar to the rock kPa strength.

3.5.2 Spread Foundations

3.5.2.1 Foundation depth

Reference is made to Section 3.2.1.

3.5.2.2 Bearing capacity and settlement

The foundation design shall take into account:

- The strength of the rock/soil
- The geometry of joints, faults and other heterogeneities in the rock mass,
- The flow and in some instances compositions of ground water
- All possible failure modes due to the above conditions.

In determining the allowable load, both absolute and differential settlements shall be considered, including time effects.

3.5.2.3 Testing

Testing shall be prescribed by the geotechnical engineer/engineering geologist, and shall be based on the load and equipment requirements. (Refer to 240-57127951: Standard for the Execution of Site Investigations).

3.6 EARTHWORKS

3.6.1 General

3.6.1.1 General References

Reference is made 240-57127953: Execution of Site Preparation and Earthworks Standard

3.6.1.2 Site Investigation

Prior to the design of an excavation, fill, soil improvement, dewatering, etc.; a site investigation shall be carried out in accordance with the latest applicable site investigation procedures. Site investigations shall include; as a minimum; intrusive ground profiling (soil profiling/core logging as necessary) groundwater intersections and groundwater ingress must be quantified, including in-situ soil testing and empirical use of laboratory data.

3.6.1.3 Excavations

For excavation at greater depth, groundwater head occurs that is higher than the bottom elevation of an excavation, therefore the stability of the base of the excavation shall be checked.

The Geotechnical investigation shall ensure the design of slopes of the excavations with a safe allowable factor of safety taking into account a possible surcharge owing to excavated material stored near the top of the slope. During construction, the Contractor shall ensure the stability of the slope/s is maintained.

3.6.1.4 Fills and Embankments

The design criteria for a fill or embankment shall be related to the intended use of the fill. Compaction of the fill, materials sources and acquisitions, and required materials grading shall be specified within the geotechnical report depending on:

- Design requirements, vs. strength (bearing capacity, slope stability), stiffness (settlement)
- Construction method
- Applied fill material

The stability and settlements of a fill or embankment shall be calculated in accordance with Sections 3.6.2 and 3.6.3.

3.6.2 Slopes

3.6.2.1 Erosion Protection

Seepage from a slope shall be prevented.

Permanent slopes shall be protected against erosion by wind or water.

Erosion protection for slopes shall comply with the latest applicable National and International Standards, and shall be confirmed with the principal engineer and geotechnical engineer prior to implementation.

3.6.2.2 Stability

3.6.2.2.1 Calculations

The long term and short term stability of a slope shall be analysed. Possible shear reduction due to (excess) pore water pressures shall be considered. Special loads, e.g. surcharges due to traffic or crane loads shall be taken into account if relevant. Consideration shall be given to the slope deformations.

Slope stability shall be assessed using the latest applicable National and International Standards, and must be verified by reliable software models.

3.6.2.2.2 Safety factors

The required safety factors shall be based on sound engineering judgement and careful analysis of:

- Consequences of a failure
- Heterogeneity of the soil conditions
- Extent of the site investigation
- Required lifetime of the slope
- Accepted slope deformation, maintenance and effects on adjacent structures
- Effects of earthquakes

Note: After failure the (residual) soil strength will be less than the original strength, which may impede (future) construction works near the failed area.

3.6.2.2.3 Seismic Activity

The analyses of natural or induced seismic activity effects on a slope shall include but not be limited to:

- Horizontal and vertical accelerations
- Increase of pore water pressures
- There is a higher risk of slip induced by agitation of fine material

3.6.3 Settlements

The following types of settlement should be analysed:

a. Consolidation of the fill material itself

b. Settlement of the subsoil:

- Elastic (undrained) compression
- (Primary) consolidation
- Secondary compression or creep

Settlement calculations shall be based on the stress distribution that is appropriate for the type of surcharge (e.g. flexible tank or rigid footing) and the type of subsoil.

The calculations shall be based on soil compressibility parameters determined by laboratory tests on samples from each compressible layer.

3.6.4 Soil Improvement

3.6.4.1 General

The following methods may be applied separately or in combination (Also see Section 3.6.4.2.):

a. Preloading

b. Drainage

c. Shallow compaction

d. Soil replacement

Other methods of soil improvement may be allowed after field trials and approval by the principal. Such methods include:

- Deep compaction
- Dynamic consolidation using a falling weight
- Stone, lime or grout columns

3.6.4.2 Review of Methods

Compaction methods shall be employed to improve the bearing capacity yield of soils at shallow depth. The compaction technique to be adopted shall be decided upon by the geotechnical engineer and the principal, and will be based on the geotechnical investigation report.

3.6.4.2.1 Preloading compaction

The design of the preloading shall include:

- Area to be preloaded
- Required weight of the preloading
- Required duration of the preloading
- Stability of the preloading
- Prediction of the settlements versus time
- Effects of vertical drainage, if relevant

During the preloading the actual settlements of the original subgrade shall be monitored. For major works, the pore water pressure in the compressible soil layers shall also be monitored.

3.6.4.2.2 Drainage Compaction

Horizontal drainage may be applied to lower the groundwater table.

Vertical drains may be installed to accelerate the consolidation of compressible soil layers. Either sand drains or plastic drains may be applied.

Sand drains shall have a minimum diameter of 0.25 m. The sand shall contain at most 10% (mass) of particles smaller than 0.063 mm and less than 3% organic matter. The drains shall be installed by jetting or drilling, not by driving.

The manufacture of plastic drains is subject to approval by the principal.

To discharge the excess water the top of the drains shall be placed in a free draining (sand) layer of at least 0.6 m thickness. The drains shall penetrate the compressible soil layers to be drained. However, the drains shall be installed to a maximum depth of 1.0 m above the top of the water-bearing layer below those compressible layers.

Drainage of groundwater from subsoil creates a cone of depression around the area being drained, it is critical that the designer be aware of this effect with regard to settlements. The removed water shall be re-introduced into the natural groundwater system at a distance away from the desired site. This prevents the possible affects to adjacent structures induced by dewatering.

3.6.4.2.3 Compaction by Soil Replacement

The bottom of the excavation shall be inspected for soft spots before backfilling is started. The backfill shall be compacted. See Section 3.5.4.2.3.

Soil replacement is the act of replacing the in-situ materials with a more stable fill material. In some instances, the fill may be constituted by a combination of natural materials as advised by the geotechnical report, while other situations may adopt artificial fills such as concrete, and/or a mixture of soil with chemical stabilizers.

The soil material backfill is then compacted according to a backfill specification, which will identify the kPa yield requirement and the plant to be used.

3.6.5 Compaction by Dewatering and Drainage

3.6.5.1 Design

For substantial dewatering systems, the design shall be based on the results of a site investigation summarized in a geo-hydrological schematization, including the following data:

- Number and type of aquifers (water-bearing strata)
- Hydraulic data per aquifer:
  - piezometric head,
  - transmissivity,
  - Hydraulic resistance of confining layers (aquicludes/aquitards/rock layers)
  - Specific yield/storage co-efficients
- Existing groundwater and surface flows
- Effects of rainfall, etc.
- Chemical content of the groundwater per aquifer.

The design shall consider time effects (non-stationary flow) and stationary flow if relevant.

For major projects the geo-hydrological model and its parameters should be verified by a pumping test.

Attention shall be paid to the possibility of rotting and clogging of drains and filters. Washing out of fines from the surrounding soil shall be prevented. Porous drains shall be installed at such a depth that the system will be continuously submerged.

After installation of wells the output of each individual well should be tested.

It shall be analysed whether emergency provisions are required in case of sudden failure of the dewatering system.

3.6.5.2 Effects on the Environment

The design shall analyse the possible environmental effects of:

- Discharging the drainage water (considering both quantity and biological/chemical contents)
- Lowering the groundwater table (slow rate of recharge, settlement, aeration of organic soils)
- Groundwater flows directed inward or outward plot or downward to deep water bearing layers (e.g. via vertical drains)

3.7 EARTH RETAINING STRUCTURES

3.7.1 Structure Types

The following types of retaining structures may be applied:

Type Ia - sheet piling

Type Ib - diaphragm walls and pile walls

Type II - gravity type or reinforced concrete retaining walls

3.7.2 Loads

The following loads shall be considered in the design if relevant:

- Hydrostatic pressures due to open water or groundwater
- Increased water pressures due to (ground) water flow
- (Excess) pore pressures
- Effective earth pressures
- External vertical or horizontal loads on or behind the retaining structures (e.g. mooring forces, traffic loads)
- Earthquakes

Also see Sections 3.1.4 and 3.1.5.

3.7.3 Sheet Piling

Sheet pile walls can be defined as continuous interlocked pile segments embedded within the soil profile to resist horizontal pressures. Sheet piling systems are viewed as “flexible” retaining systems.

Sheet pilings used for retaining walls are classified as anchored sheet piles or cantilevered sheet piles. Sheet piles are made of various materials such as wood, concrete, steel or aluminium.

3.7.3.1 Applications of Sheet Piles

- Cantilevered Sheet Piles
  - Primarily used for protection of permanent or temporary excavations.
  - Used for heights of about 6m.
- Anchored Sheet Piles
  - Used for heights less than 6m.
  - Anchoring of the sheet piles to less penetration depth cause less moment to the sheet pile. Anchored walls are always pre-stressed; this removes slacks from the system. The anchors place the entire soil mass between anchors and wall in compression, thereby creating a very large gravity wall. Subsidence may occur in anchoring systems due to:
- Flow of cohesionless material into the excavation made for installation
- Caving of anchor holes prior to grouting
- Downward movement of the wall due to the vertical component of the anchor forces

3.7.3.2 Limitations

- Sheet piling sections can rarely be used as part of a permanent structure
- Installation is difficult in areas where soils contain boulders or cobbles
- Excavation shapes are determined by the sheet pile section and interlocking elements
- Driving of sheet piles may cause disturbances
- Adjacent settlements may be caused by vibrations during installations

3.7.3.3 Types of Sheet Piling

There are two types “Z” type and “U” type:

- Wooden Sheet Piles: used for temporary light structures to prevent cave-ins. Due to degradation of wood over time, the wood occurs fully encapsulated, or treated with chemicals like chrome, aluminium or steel plating.
- Concrete Sheet Piles: used for bulkheads in either fresh or saltwater. However, precast sheet piles are heavy and produce large disturbances during driving.
- Steel Sheet Piles: popular due to strength, ease of handling and ease of construction; and available in various cross section shapes.
- Aluminium Sheet Piling: the material is durable and resistant to corrosion, and can be applied in similar cross sections to steel sheet piles.
- Vinyl Sheet Piles: used for heavy loads and marine applications.

3.7.4 Corrosion

The corrosiveness potential of surface water, ground water, soil and backfill material shall be investigated. See Section 3.3.4.3.

3.7.5 Diaphragm Walls and Pile Walls

The design of diaphragm walls (reinforced concrete slurry walls) and pile walls is subject to approval by the principal.

3.7.6 Gravity Type or Reinforced Concrete Retaining Walls

In the structural design of this type of rigid retaining structures it shall be assumed that the acting pressure is the effective earth pressure at rest (being at least 50% of the effective vertical pressure) plus the (ground) water pressures.

The overall stability (overturning moment) as well as the maximum bearing pressure shall be calculated assuming active earth pressure plus (ground) water pressure as a minimum load. The safety factor against overturning shall be at least 1.5. The allowable bearing pressure shall be calculated according to Section 3.2.

Moreover an overall stability analysis according to Section 3.6.2.2 shall be carried out.

3.8 GROUND IMPROVEMENT

Ground Improvement techniques may be adopted as permanent or temporary:

- Permanent – may be defined as any ground improvement technique adopted for a time period of greater than 2 years.
- Temporary - may be defined as any ground improvement technique adopted for a time period of less than 2 years.
- Ground Anchors – a device that is designed to support structures used in geotechnical and construction applications, may be used as either temporary or permanent applications.
- Rock Bolts and Dowels - a long anchor bolt used for stabilising rock excavations (tunnels and rock cuts) and transfers loads from the “unstable” exterior to the confined interior of the rock mass.
- Soil Nailing – a construction technique used as a remedial measure to treat unstable natural soil slopes. Soil nailing can be used to increase the factor of safety of the overall slope design.

Ground improvement designs by means of ground anchors, rock bolts and soil nailing shall be done in accordance to the latest applicable national and international standards, and latest software implementation. These are all highly specialised design activities and must be conducted by a specialist geotechnical engineer or contractor.

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This document has been seen and accepted by:

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5. REVISIONS

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6. DEVELOPMENT TEAM

The following people were involved in the development of this document:

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- Bruno Marrai

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- N/A