CHAPTER 4
FOUNDATION

4.1. General

The function of a foundation is to transfer the structural loads from a building safely into the ground. The structural loads include the dead, superimposed and wind loads. To perform the function, the foundation must be properly designed and constructed. Its stability depends upon the behaviour under load of the soil on which it rests and this is affected partly by the design of the foundation and partly by the characteristics of the soil. It is necessary in the design and construction of foundation to pay attention to the nature and strength of the materials to be used for the foundations as well as the likely behaviour under load of the soils on which the foundation rests.

4.2. Soil Characteristics

It is convenient to categorise soils and their properties according to their particle size as shown in Table 4.1.

The soil conditions in Singapore are basically made up of the following ten geological formations:

1) Sajahat Formation (S)
2) Gombak Norite (GN)
3) The Paleozoic Volcanics (PV)
4) Bukit Timah Granite (BT)
5) The Jurong Formation
6) Fort Canning Boulder Bed (FC)
7) The Old Alluvium (OA)

8) The Huat Choe Formation (HC)
9) The Tekong Formation (T)
10) The Kallang formation (K)
    - Marine Member (Km)
    - Alluvial Member (Ka)
    - Littoral Member (Kl)
    - Transitional Member (Kt)
    - Reef Member (Kr)

The four major ones are: Bukit Timah Granite (BT), Jurong Formation, Old Alluvium (OA) and Kallang formation (K) as shown in Figure 4.1 [1].

Bukit Timah Granite

The Bukit Timah Granite is distributed in the central part of the island such as Bukit Timah, Thomson, Sembawang, Mandai, Bukit Panjang and Upper Changi areas. A typical cross section of the formation is shown in Figure 4.2(a). The weathered granite residual soil is of sandy clay or silt nature and consists of between 25% to 65% silt and clay-sized particles with increasing stiffness and percentage of coarser fraction with depth. Large boulders are often encountered within this weathered profile. The residual granite soil covers one-third of the surface of central Singapore. At some low-lying areas, a layer of alluvium or marine clay may overlie the granite formation.

Jurong Formation

The Jurong formation covers the western and southern coastal areas. This formation consists of sedimentary rocks formed in the Triassic period. Mudstone, sandstone and shales can be found interbedded in this formation at depth ranging from 5 m to 45 m. A typical cross section of the formation is shown in Figure 4.2(b).

Old Alluvium Formation

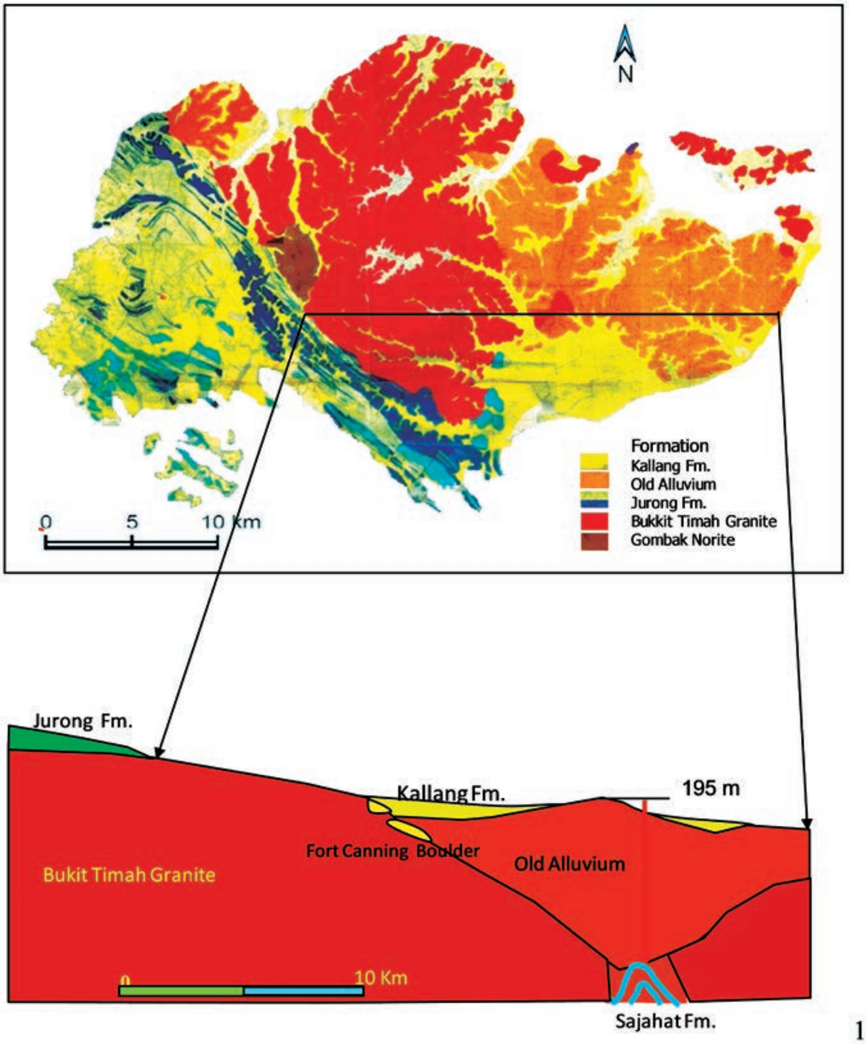
This formation lies on the north-eastern and north-western part of the island. It is a product of heavily weathered sedimentary rocks consisting

Table 4.1. Soil characteristics and bearing capacities [3].

Subsoil types	*Condition of subsoil*	*Means of Field Identification*	*Particles size range*	*Bearing capacity kN/m²*	*Minimum width of strip foundations in mm for total load in kN/m of loadbearing wall of not more than:*					
					20	*30*	*40*	*50*	*60*	*70*
Gravel	Compact	• Require pick for excavation. 50 mm peg hard to drive more than about 150 mm. • Clean sands break down completely when dry. Particles are visible to naked eye and gritty to fingers. Some dry strength indicates presence of clay.	Larger than 2 mm	> 600	250	300	400	500	600	650
Sand			0.06 to 2 mm	> 300						
Clay	Stiff	• Require a pick or pneumatic spade for removal. • Cannot be moulded with the fingers. • Clays are smooth and greasy to the touch. Hold together when dry, are sticky when moist. Wet lumps immersed in water soften without disintegration.	Smaller than 0.002 mm	150–300	250	300	400	500	600	650
Sandy clay			See Sand and Clay	150–300						
Clay	Firm	• Can be excavated with graft or spade. • Can be moulded with strong finger pressure.	See above	75–150	300	350	450	600	750	850
Sandy clay			See Sand and Clay	75–150						
Gravel	Loose	• Can be excavated with a spade. • A 50 mm peg can be easily driven.	See above	< 200	400	600	For loading of more than 30.0 kN/m run on these types of soil, the necessary foundations do not fall within the provisions of Approved Document A, Section 1E from which these figures are taken. Pad foundations generally and surface rafts are designed using the bearing capacities for soils given in this Table.			
Sand			See above	< 100						
Silty sand			See Silt and Sand	May need to be assessed by test						
Clayey sand			See Clay and Sand	ditto						
Silt	Soft	• Readily excavated. • Easily moulded in the fingers. • Silt particles are not normally visible to the naked eye. Slightly gritty. Moist lumps can be moulded with the fingers but not rolled into threads. Shaking a small moist pat brings water to surface which draws back on pressure between fingers. Dries rapidly. Fairly easily powdered.	0.002 to 0.06 mm	< 75	450	650				
Clay			See above	< 75						
Sandy clay			See Sand and Clay	May need to be assessed by test						
Silty clay			See Silt and Clay	ditto						
Silt	Very soft	• A natural sample of clay exudes between the fingers when squeezed in fist.	See above	ditto	600	850				
Clay			See above	ditto						
Sandy clay			See Sand and Clay	May need to be assessed by test						
Silty clay			See Silt and Clay	ditto						
Chalk	Plastic	• Shattered, damp and slightly compressible or crumbly.	–	–	Assess as clay above					
Chalk	Solid	• Requires a pick for removal.	–	600	Equal to width of wall					

Sands and gravels: In these soils the permissible bearing capacity can be increased by 12.5 kN/m² for each 0.30 m of depth of the loaded area below ground level. If groundwater-level is likely to be less than the foundation width below the foundation base the bearing capacities given should be halved. The bearing capacities given for these soils assume a width of foundation around 1.00 m but the bearing capacities decrease with a decrease in width of foundation. For narrower foundations a reduced value should be used: the bearing capacity given in the table multiplied by the width of the foundation in metres.

Geological Map of Singapore (After DSTA, 2009)



Generalized cross section of the Central to East of
Singapore (Approximate Scale)

Data based on Geology of Singapore (DSTA, 2009)
and Field data.

Figure 4.1. The major geological formations in Singapore (Courtesy: Naing Maw Than, "Notes on Geology of Singapore").

	SOIL PROFILE	N-VALUE	MOISTURE (%)
	Clayey Silt Fill	20-30	20-40
	Stiff to very stiff sandy clayey silt	40-60	20-40
	Weathered granite	>80	

Figure 4.2(a). Typical soil profile of Bukit Timah Granite.

	SOIL PROFILE	N-VALUE	MOISTURE (%)
	Medium Stiff Sandy Silty Clay	10-20	20-40
	Soft Sandy Silty Clay	10-20	20-40
	Very Stiff Sandy Silty Clay	20-40	20-40
	Hard Silty Clay	60-80	20-40
	Very Dense Clayey Silt	>100	20-40
	Silty Clay with Shale	>100	20-40

Figure 4.2(b). Typical soil profile of Jurong Formation.

	SOIL PROFILE	N-VALUE	MOISTURE (%)
	Sandy silty clay	10-20	20-40
	Stiff silty clay	20-30	20-40
	Hard silty clay (with some sand)	40-50	20-40
	Very dense cemented silty sand	>100	10-20

Figure 4.2(c). Typical soil profile of Old Alluvium Formation.

	SOIL PROFILE	N-VALUE	MOISTURE (%)
	Clayey fill	10-20	20-30
	Soft to firm marine clay (upper member)	<3	60-80
	Stiff clay	20-30	
	Firm marine clay (lower member)	<5	40-60
	Medium to dense cemented clayey sand	>100	20-30

Figure 4.2(d). Typical soil profile of Kallang Formation.

primarily of clayey to silty coarse sand with layers of silty clay as shown in Figure 4.2(c). It is often thick, in excess of 30 m, with a total thickness of 195 m having been recorded [2].

Kallang Formation

This formation consists of soils with marine, alluvial, littiral and estuarine origins, which covers much of the coastal plain, immediate off-shore zone and deeply incised river valleys, which penetrates to the centre of the island. Basically, the formation comprises of two layers, the upper and lower marine clay, separated by a thin layer of stiff silty clay as shown in Figure 4.2(d).

4.3. Foundation Systems

Due to the greater compressibility, cohesive soils suffer greater settlement than cohesionless soils. In Singapore, with the rapidly increasing urbanisation and industrialisation, many structures are constructed on poor soils.

Figure 4.3 shows the various types of foundation system used for buildings in the Central Business District (CBD) in Singapore. The types of foundation system used by buildings are quite diversified, indicating

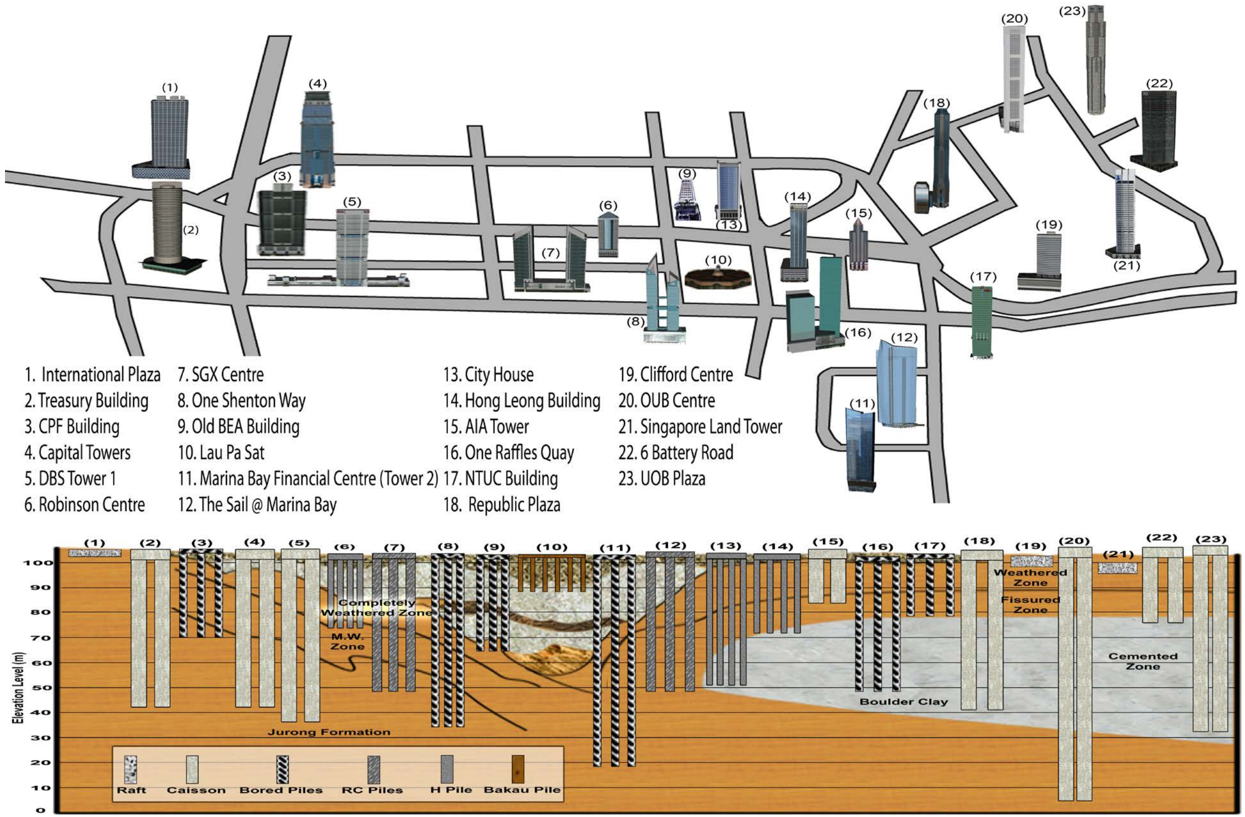


Figure 4.3. Foundation system of some major buildings in the Central Business District (CBD).

that these systems are feasible and suitable to their respective buildings, at least at their time of construction.

The following ground materials are commonly encountered in a typical boring investigation: (a) fill layer, (b) Huat Choe Formation, (c) bouldery clay, (d) marine clay.

Fill Layer: The fill layer consists mainly of loose materials of sand, bouldery sand and boulders with clay. The colour of the fill is light brown to dark brown and is highly permeable to water.

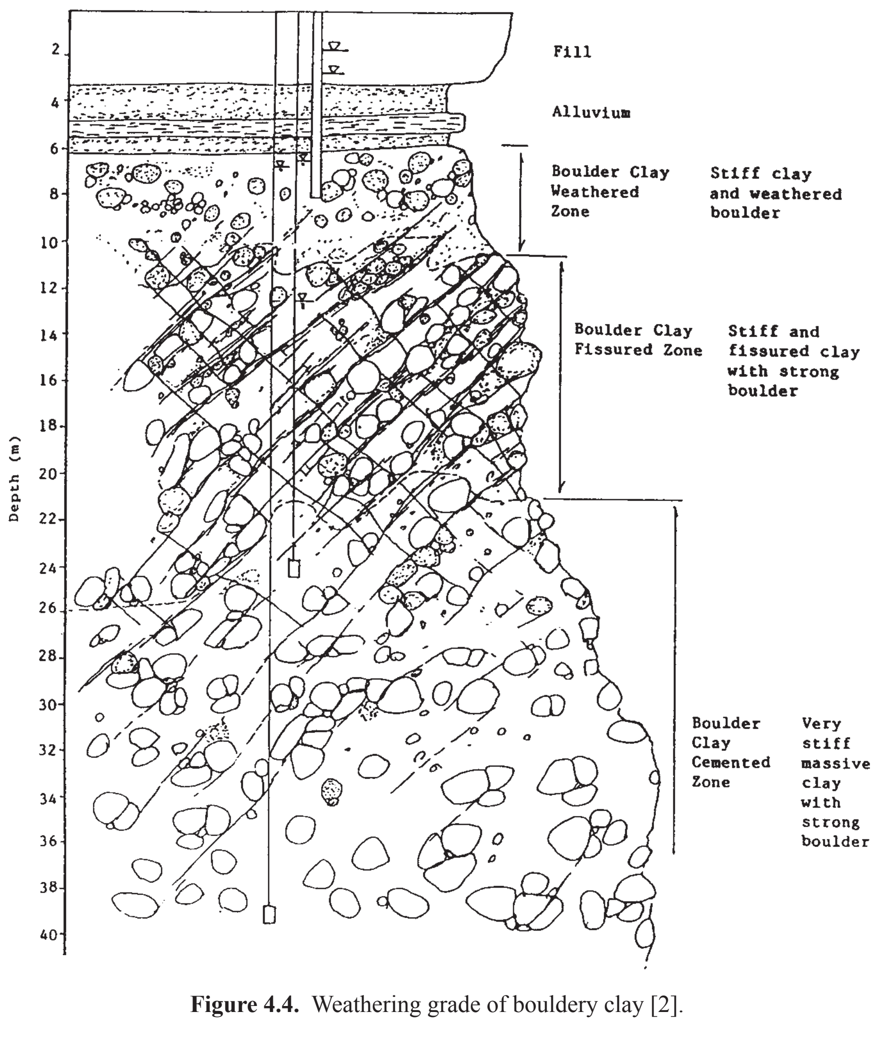


Figure 4.4. Weathering grade of bouldery clay [2].

Huat Choe Formation: Huat Choe Formation consists mainly of hard sandy silt and is light greyish white in colour. The formation is not porous and water cannot seep through the formation easily.

Bouldery clay: This consists mainly of strong rounded boulders of varying size and stiff soils. The bouldery clay layer is usually composed of a few zones according to its weathering grade. An example is shown in Figure 4.4 with its properties shown in Table 4.2.

Marine Clay: Marine clay is kaolinite rich, pale-grey to dark blue-grey, soft silty clay, with sandy, silty, peaty and shelly fragments. The fragmented shells indicate its marine origin. The *in situ* moisture content is close to the liquid limit and the cohesive strength is low. The marine clay does not become siltier with depth, hence retaining its essential characteristic of 65% to 70% clay throughout [3]. It is usually consolidated with average shear strength of 10 kN/mm^2 and 40 kN/mm^2 for the upper and lower marine member respectively.

Geologists believe that marine clay is formed from materials brought down by the river and with the retreat of the ice in the Pleistocene period, the land rose because of the consequent relief in load. Weathering and denudation then took place and the materials formed were deposited by the rivers in swamps and estuaries with the

Table 4.2. Contents of bouldery clay.

Weathered Zone	Most of the boulders are weathered and decomposed into weakly cemented sand or friable sandstone. Matrix is stiff clay and weakly cemented. There are no fissures or open discontinuities in this zone.
Fissured Zone	Some boulders are weathered and turn into weak sandstone but some of the boulder reserves are still very strong in compressive strength. Most of the boulders show traces of weathering in colour and strength. Matrix is very hard, massive clay and have many fissures and open discontinuities in this zone. Near the open discontinuities, the clay appears weakened by swelling. Groundwater seeps out of the fissures and discontinuities are discovered.
Cemented Zone	Boulders are white; strong sandstone with thin weathering zone at the surface area. Matrix is hard and dense massive clay with some fine gravel. There are some shear zones and slicken sides in the matrix but all fissures are closed and permeability is considered low.

heavier particles i.e. sand and gravels at the bottom and the light ones such as clays above. These sediments were subsequently covered by sea water as a result of the submergence of the landmass under the seas. With further emergence of the land and regression of the seas, more erosion and deposition took place. The soft marine clay is generally blue or grey in colour. Its natural moisture content is high at about 80%–100% depending on the depth [4]. The soil is very compressible and has a very low shear strength. Figure 4.5 shows an example of foundation works on marine clay.

Figure 4.5. Foundation works on marine clay.

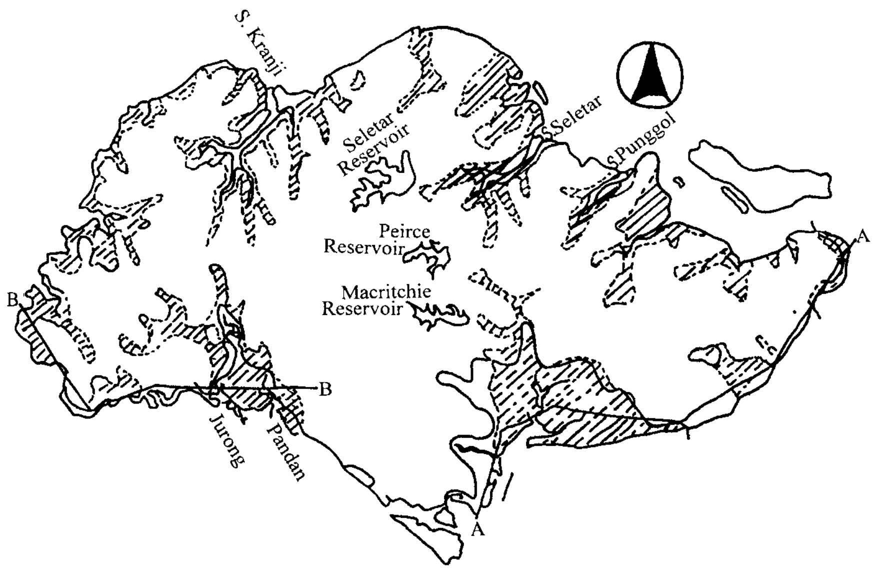


Figure 4.6. Location showing marine clay areas.

In Singapore, soft marine clay covers a large area mainly along the estuaries of the present river system (Figure 4.6). The clay varies in thickness from place to place, up to about 30 m thick along Nicoll Highway.

The low shear strength coupled with high compressibility of the thick marine clay suggests that deep foundation is necessary. Problems anticipated when dealing with these kinds of soil include ground heave, adjacent ground surface settlement, long term settlement and low bearing capacity. Negative skin friction may also develop [4, 5].

4.4. Types of Foundation

On the selection of a suitable foundation system for a building, various factors must be taken into consideration. Among them are soil conditions, load transfer pattern, shape and size of the building, site constraints, underground tunnels and/or services, environmental issues, etc. [6]. There are two basic types of foundations:

- Shallow foundations: those that transfer the load to the earth at the base of the column or wall of the substructure.

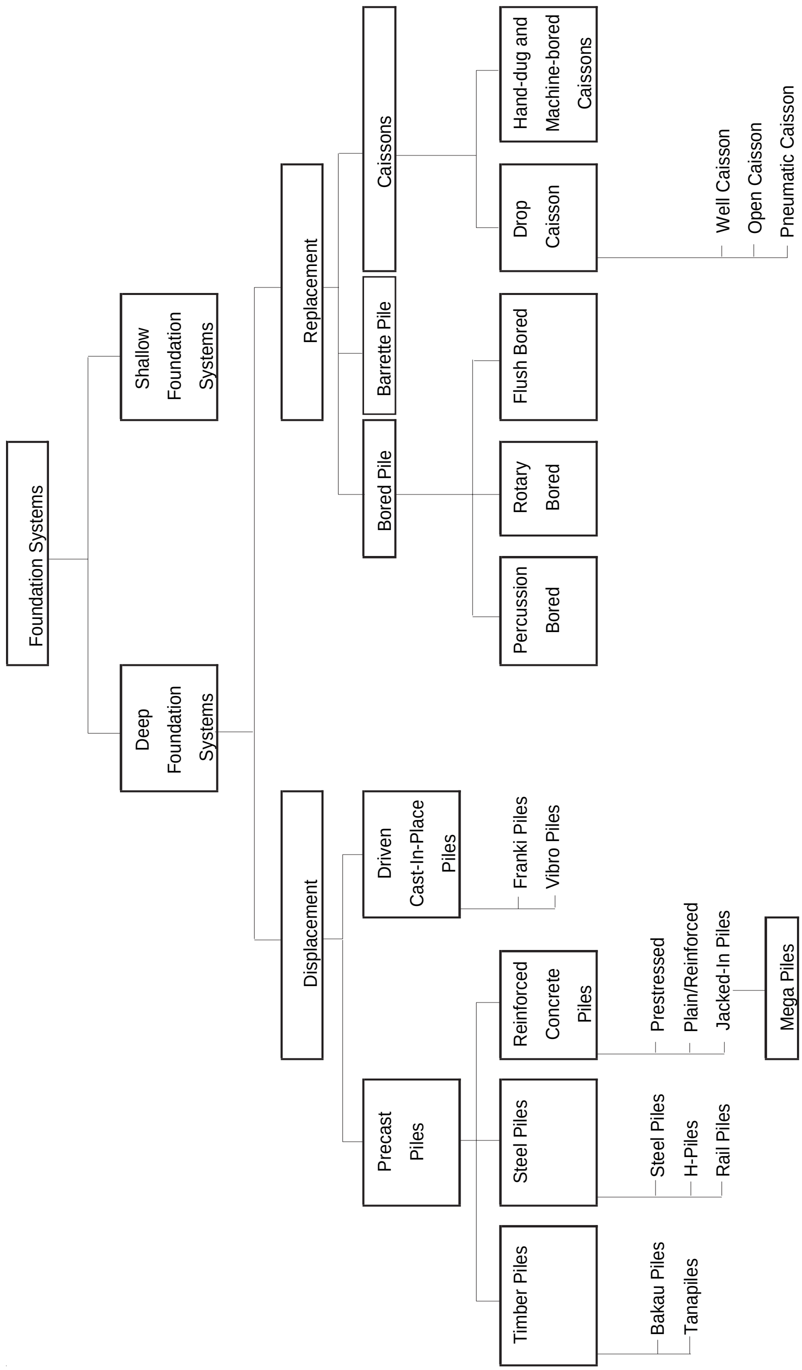


Figure 4.7. General classification of foundation systems.

– Deep foundations: those that transfer the load at a point far below the substructure.

A general classification of foundation systems is shown in Figure 4.7.

4.5. Shallow Foundation

Shallow foundations transfer loads in bearing close to the surface. They either form individual spread footings or mat foundations, which combine the individual footings to support an entire building or part of it. The two systems may also act in combination with each other, for example, where a service core is seated on a large mat while the columns are founded on pad footings [7].

Spread footings are divided into isolated footings (e.g. column footings), strip footings (e.g. wall footings), and combined footings. Figure 4.8 shows some common shallow foundations. A column or isolated footing is a square pad of concrete transferring the concentrated load from above, across an area of soil large enough that the allowable stress of the soil is not exceeded. A strip footing or wall footing is a continuous strip of concrete that serves the same function for a loadbearing wall.

In situations where the allowable bearing capacity of the soil is low in relation to the weight of the building, column footings may become large enough so that it is more economical to merge them into a single mat or raft foundation that supports the entire building. A mat foundation is basically one large continuous footing upon which the building rests. In this case, the total gross bearing pressure at the mat-soil interface cannot exceed the allowable bearing strength of the soil. Mat or raft foundation, however, is not suitable for tall buildings where the soil condition is not adequate. The highly concentrated and eccentric column loads means the thickness would need to be very large (often > 2 m) [8–10].

Examples of tall buildings built on rafts in the Central Business District (CBD) in Singapore include Shell Tower, Hitachi Tower, Ocean Building, Raffles City Complex and the Tung Centre. They are constructed in bouldery clay. The competitive advantage for using raft foundation at that time include:

- The presence of large and strong boulders making the use of driven precast concrete or steel piles difficult.
- The high bearing capacity of the bouldery clay and the small settlements due to the low compressibility of the very stiff or hard silty clay matrix.
- Lower contruction cost due to the simpler construction method and shorter construction time, compared with caissons or large diameter bored piles.

Figure 4.8(b) shows the cross section through the 128 m Hitachi Tower and 226 m Swissôtel The Stamford, including the raft.

The Hitachi Tower consists of a 33-storey tower block, a 4-storey podium and a 3-storey basement. The 2.8 m thick raft, which is 40 m × 68 m in plan, is located 16 m below the ground surface. The top and bottom steel reinforcement is 0.96% in both directions. There are two layers of Y32 reinforcing bars with 200 mm spacing and three layers of Y40 reinforcing bars with 200 mm spacing at the top and bottom in both directions of the raft. The average gross unit load transmitted to the foundation is 540 kPa for the tower and 90 kPa for the podium. The Swissôtel The Stamford consists of a 76-storey tower block, with a

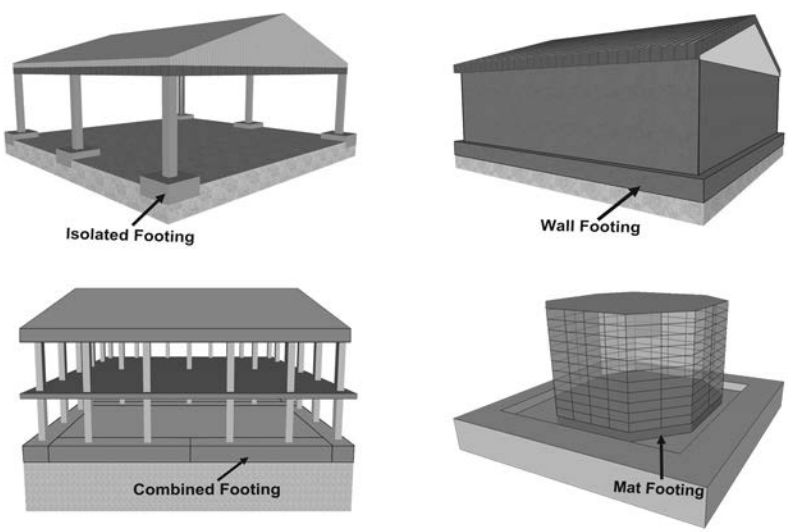


Figure 4.8(a). Types of shallow foundation.

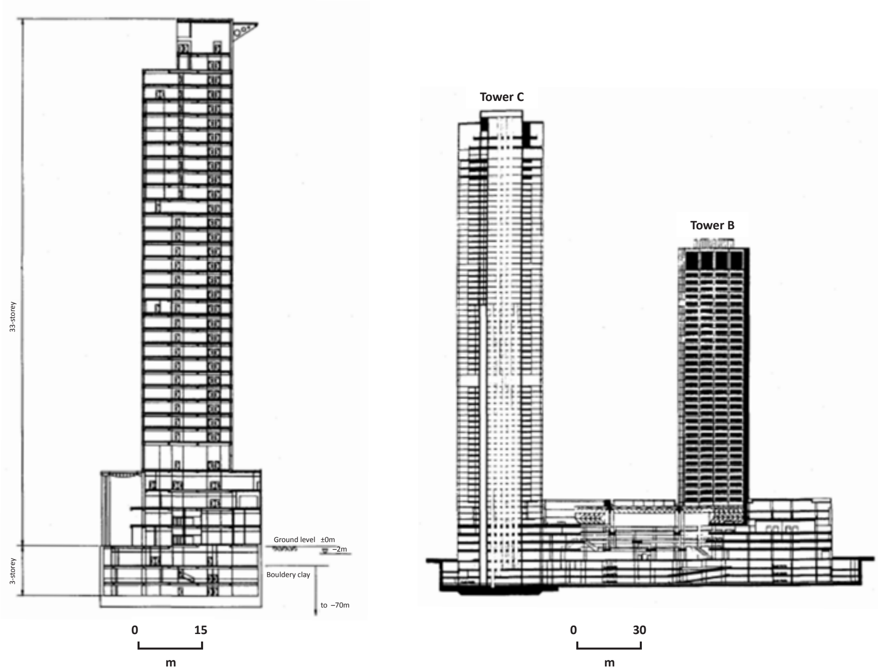


Figure 4.8(b). Section showing raft foundation of Hitachi Tower (left) and Swissôtel The Stamford — Tower C (right) [11].

3-storey basement. The 4.9 m thick raft, which is 58 m × 61 m in plan, is located 18 m below the ground surface. It is reinforced by one layer of Y38 at 200 mm spacing and by two layers of Y38 at 300 mm spacing at the top and bottom of the raft in both directions. An additional one or two layers of Y38 reinfrocements at 300 mm spacing have been placed under the heavily loaded columns. The raft has a post-tensioned section (1.6 m thick) to distribute the high loads. The average gross unit load transmitted to the foundation is 439 kPa [11].

4.6. Deep Foundation

Deep foundations are used when adequate soil capacity is not available close to the surface and loads must be transferred to firm layers substantially below the ground surface. The common deep foundation systems for buildings are caissons and piles as shown in Figure 4.9 [11–19].

Figure 4.10 illustrates the various deep foundation types used according to the bearing layers. The common types of deep foundation including caisson, bored pile, barrette pile, driving reinforced concrete and steel piles are discussed in the following sections.

4.7. Caisson

Examples on the use of caissons for tall buildings include UOB building (12 caissons), OUB building (7 caissons), AIA Tower (65 caissons), DBS Building (4 caissons), Republic Plaza (14 caisson), Capital Tower (6 caissons) etc. There have also been cases where the bored pile method was initially specified but was later switched to the use of caisson. Among the reasons are difficulties in boring, increase in superimposed loads, requirement for speed of construction, etc.

The word "caisson" comes from French "caissee" meaning a chest or case. It was originally used to refer to a water-tight chamber within which foundation work could be carried out underwater. In common usage, it has also come to mean a type of deep foundation unit.

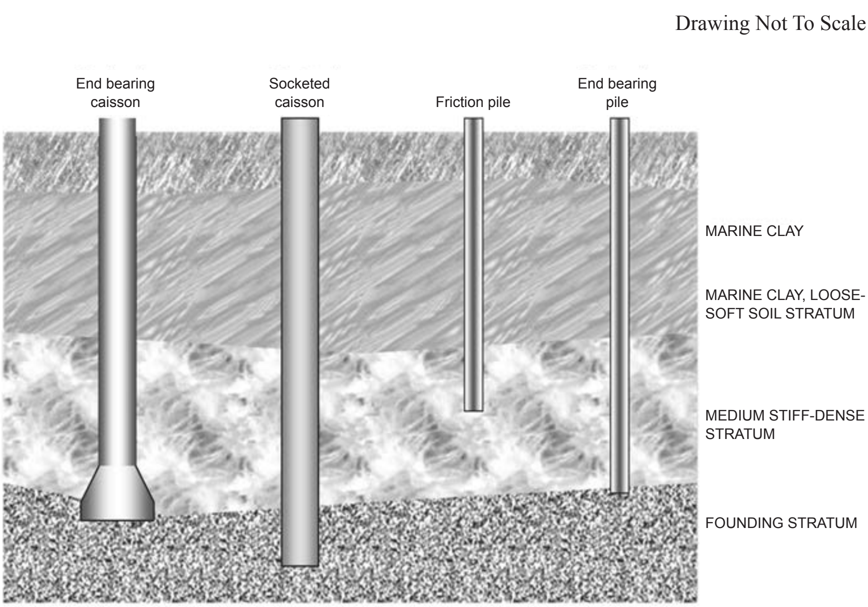


Figure 4.9. The common types of deep foundations [11].

Condition	Type of Pile	RC Pile			Spun Concrete Pile			H-Steel Pile			Steel Pile			Bored Pile				Caisson	
		250	350	450	Ø250	Ø350	Ø450	H300x 300	H388x 351	H388x 402	Ø304	Ø406.4	Ø508	Ø800	Ø1000	Ø1200	Ø1500	Ø3000	Ø5000
Soil Layer for Piling	Soft Clay	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	Loose Sand	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	ρ	ρ	ρ	ρ	ρ	ρ
	Gravel	X	ρ	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	Boulder	X	X	X	X	X	X	X	X	X	X	X	X	ρ	ρ	ρ	ρ	♥	♥
	High Permeable Layer	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	ρ	ρ
	Inclined Hard Layer	X	X	X	X	X	X	X	X	X	X	X	X	ρ	ρ	X	X	♥	♥
Capability of Penetration by Pile	N > 30	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	N > 40	ρ	♥	♥	ρ	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	N > 50	X	X	ρ	X	X	ρ	ρ	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	N > 100	X	X	X	X	X	X	X	ρ	ρ	X	ρ	ρ	♥	♥	♥	♥	♥	♥
	N > 200	X	X	X	X	X	X	X	X	X	X	X	X	ρ	ρ	ρ	ρ	♥	♥
	Strong Rock	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	ρ	ρ
Length of Pile	Less 20m	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	20-30m	ρ	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	30-40m	X	ρ	♥	X	♥	♥	ρ	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥	♥
	40-50m	X	X	ρ	X	X	♥	X	ρ	♥	X	♥	♥	♥	♥	♥	♥	♥	♥
	50-60m	X	X	X	X	X	X	X	X	ρ	X	X	ρ	♥	♥	♥	ρ	♥	♥
	More 60m	X	X	X	X	X	X	X	X	X	X	X	X	ρ	ρ	ρ	X	♥	♥

♥ Suitable
X Not Suitable
ρ Possible

Figure 4.10. Types of deep foundation according to the bearing layer.

A caisson is a shell or box or casing which, when filled with concrete, will form a structure similar to a cast-in-place pile but larger in diameter. A caisson is similar to a column footing in that it spreads the load from a column over a large enough area of soil that the allowable stress in the soil is not exceeded. It differs from a column footing in that it reaches through strata of unsatisfactory soil beneath the substructure of a building until it reaches a satisfactory bearing stratum such as rock, dense sands and gravels or firm clay (see Figure 4.9). Over the years, however, the term has come to mean the complete bearing unit [20–25].

Factors affecting the choice of using caisson piles include:

(a) On site where no firm bearing strata exists at a reasonable depth and the applied loading is uneven, making the use of a raft inadvisable.
(b) When a firm bearing strata does exist but at a depth such as to make a strip, slab or pier foundation uneconomical.
(c) When pumping of groundwater would be costly or shoring to excavation becomes too difficult to permit the construction of spread foundation.
(d) When very heavy loads must be carried through water-logged or unstable soil down to bed rock or to a firm strata, and having large number of piles with large pile caps are not economical.
(e) When the plan area of the required construction is small and the water is deep.

Bored Caisson: A bored caisson is one in which a hole of the proper size is bored to depth and a cylindrical casing or caisson is set into the hole.

This is the common type of caisson used for building construction in Singapore (see Figure 4.3). It is basically a concrete-filled pier hole for the support of columns. In Singapore, bored caissons of diameters ranging from 600 mm to 6 m are common. The hole can be bored by using a bucket drill, mini excavator or hand-dug (Figures 4.11 and 4.12).

A bored caisson depending on the size may be hand-dug or excavated with the use of mini excavators. In both cases, excavation takes place inside the caisson shafts instead of occupying additional space as compared to that of the conventional bored pile methods. It requires less

Figure 4.11. Closely spaced hand-dug caissons (ø ≈ 600 mm).

Figure 4.12(a). 12 mechanically-dug caissons (ø ≈ 6 m).

Figure 4.12(b). Construction of six bored caissons (ø ≈ 6 m).

Figure 4.12(c). Two mobile gantry cranes each serving three caissons slide along tracks cast on the concrete floor. Note safety nets on the handrails, artificial ventilators and lighting in each caisson.

Drawing Not To Scale

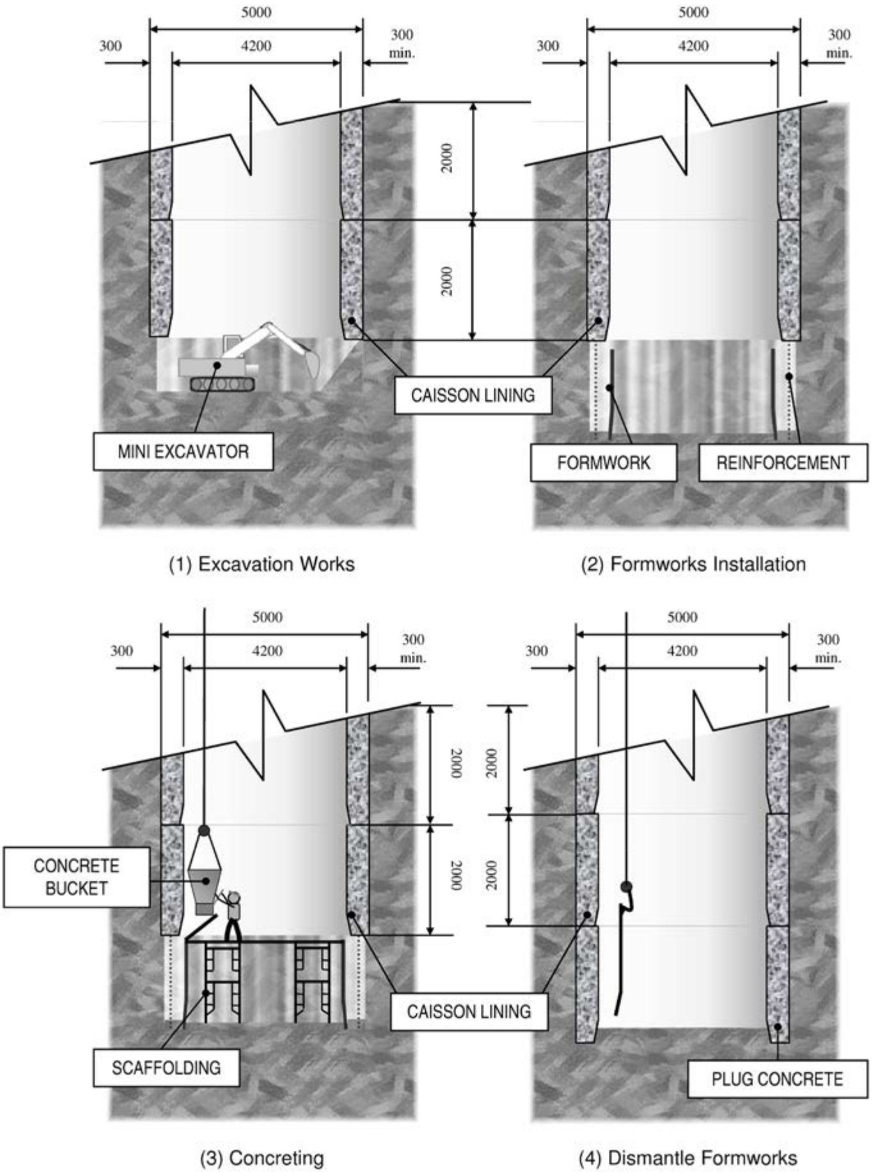
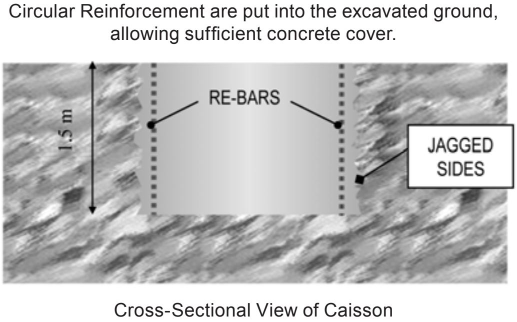


Figure 4.13. Construction sequence of a typical caisson lining.

space and hence enables a tidier site. As excavation is carried out bit by bit within a shaft, vibration, noise and dust can be reduced. Each stage of excavation is followed immediately by the casting of the concrete lining to resists the surrounding soil and water penetration.

The work sequence of bored caissons is generally as follows:

- Establish level and position of caissons.
- Excavate to a depth of 1.5 m by means of mini excavators. Workers are lowered to excavate manually in hard to reach places (Figure 4.13).



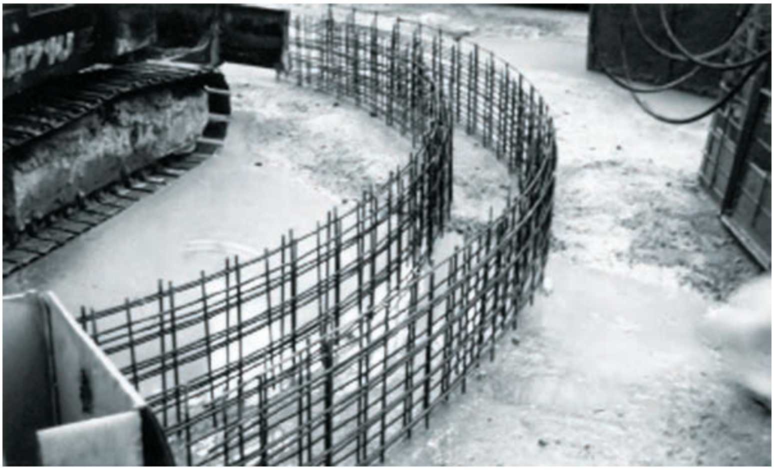
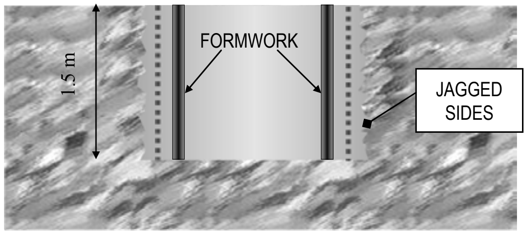


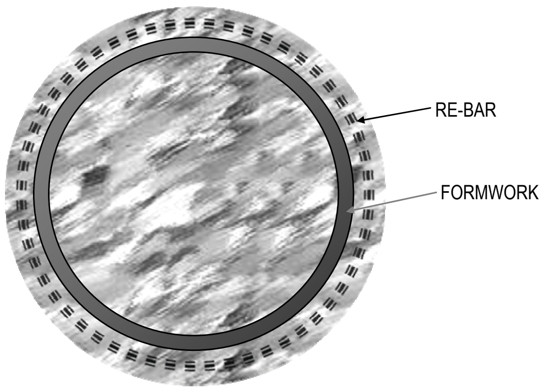
Figure 4.14. A section of the circular reinforcement mesh for the caisson lining.

- Lay circular reinforcement mesh for the caisson lining (Figure 4.14).
- Erect steel ring wall formwork (Figure 4.15). There are three segments of circular steel formwork connected with bolt and nut and locking pin. For the first caisson lining, assemble the segments on ground and hoist down for fixing.
- Cast concrete with hoisting buckets sequentially round the ring wall and progressively upward.
- Excavate further 1.5 m downward, lay reinforcement mesh. Detach formwork from the cast concrete by removing the locking pin and the use of chain block.
- Lower down the formwork to the next lower position by the use of hoisting crane and chain block for positioning and alignment.
- Fix formwork slightly inclined outward to enhance concrete placement (Figure 4.16).
- Place concrete leaving a gap to be grouted later (Figure 4.17).
- Material handling using gantry crane, mobile crane etc. (Figure 4.18).
- Provide adequate ventilation and lighting (Figure 4.19).
- Complete caisson lining to founding level.
- Carry out jack-in-test when required to assess the skin frictional resistance by inserting jacks in gap between two linings (Figure 4.20(a)). Apply jacking until the target load capacity is reached. Monitor pressure and movement using pressure gauges, level meters, strain and dial gauges etc. (Figure 4.20(b)). Figure 4.21 shows an example of the alignment of hydraulic jacks.
- Monitor the dewatering system. Control groundwater pressure around and beneath the caisson excavation as required. Provide instrumentation to monitor pore pressure changes and ground settlement when required. Provide recharge wells when necessary.
- Erect caisson reinforcement. Arrange concreting platform (Figure 4.22). Cast caisson through tremie pipes in one operation. Provide adequate vibration. Monitor temperature to avoid micro cracking due to the possible high heat of hydration. Cast concrete to the required height (Figure 4.23).

The same principles apply to small caissons, except that the whole operation has to be carried out manually (Figures 4.24 and 4.25).



Cross-Sectional View of Caisson



Plan View of the caisson

Figure 4.15. Erection of steel ring wall formwork.

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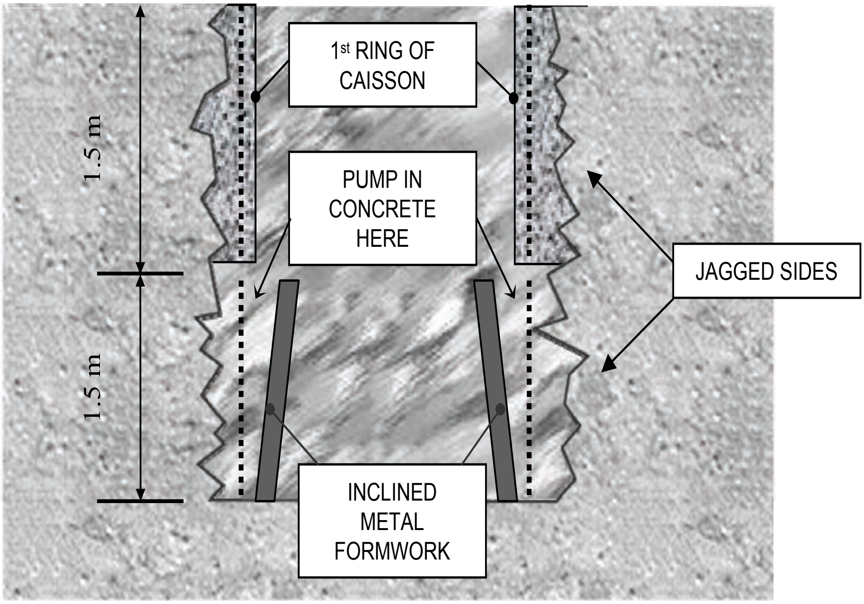

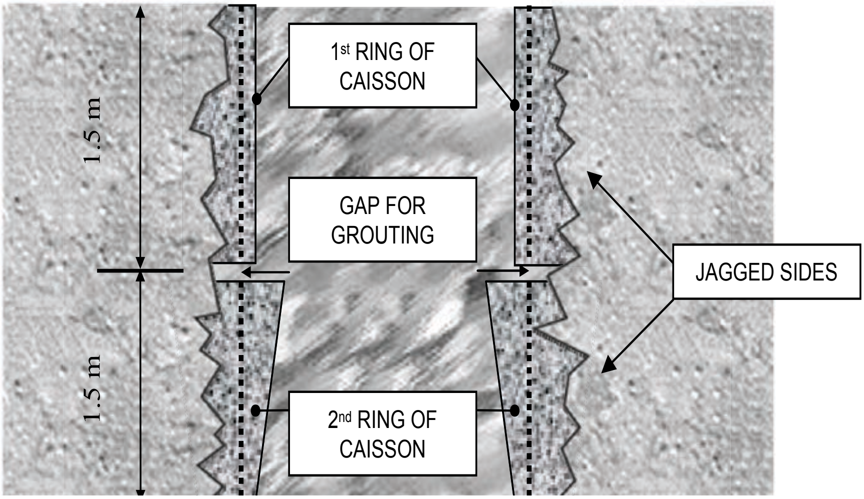

Figure 4.16. Caisson lining for subsequent layers.

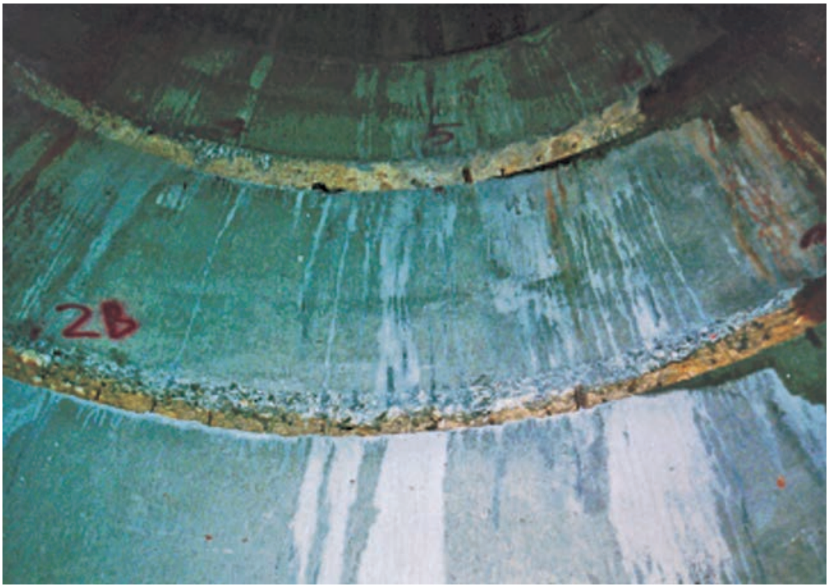

Figure 4.17(a). Completed caisson lining with a gap between each layer.

Figure 4.17(b). A closer view of the gap. Note the pins and starter bars.

Figure 4.18(a). Lowering a mini excavator using the gantry crane.

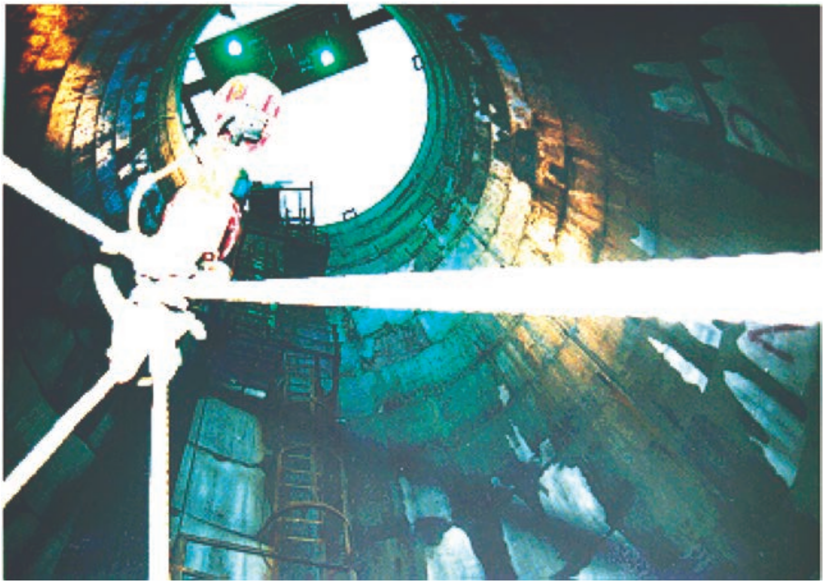

Figure 4.18(b). Material handling using a gantry crane, mobile crane, etc.

Figure 4.19(a). Provision of adequate artificial ventilation through a duct. Note the staircase on the right.

Figure 4.19(b). Provision of a blower.

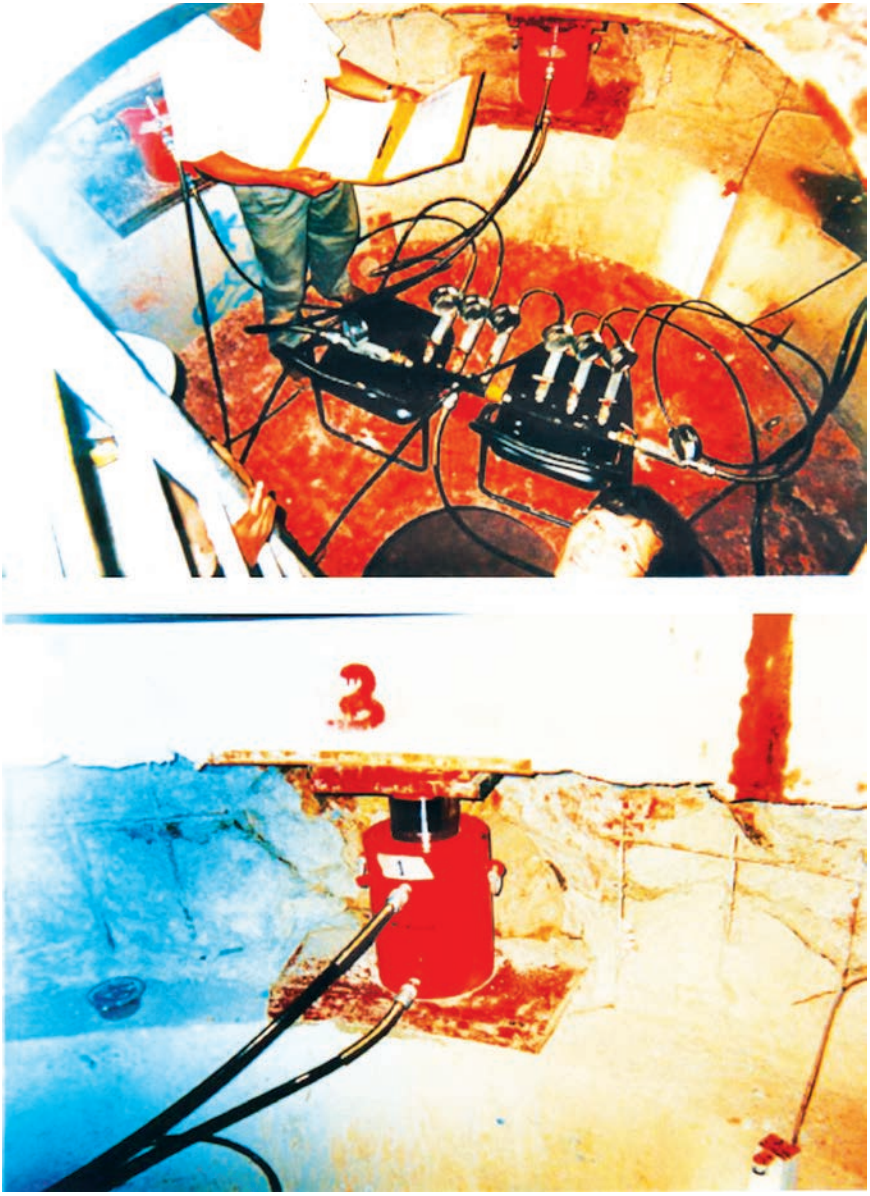

Figure 4.20(a). Hydraulic jacks placed in between two caisson rings.

Figure 4.20(b). Monitor displacement using dial gauges.

Gow Caisson: Figure 4.26 shows the construction sequence of a gow caisson. The largest cylinder is first sunk by excavating below the cutting edge and driving the cylinder down. After the first cylinder is positioned, the second with a smaller diameter is sunk in the same manner and others in succession to the desired depth.

A bell is excavated if the soil conditions permit. After the excavation is completed, the bell is filled with concrete and the cylinders are withdrawn as concreting proceeds until the pier is completed.

Socketed Caisson: A socketed caisson is one that is drilled into rock at the bottom, rather than belled. Its bearing capacity comes not only from its end bearing, but from the frictional forces between the sides of the caisson and the rock as well (Figure 4.9). A steel pipe is driven in concurrently with augering. When the bedrock is reached, a socket is churn-drilled into the rock that is slightly smaller in diameter than the caisson shell. The unit is then reinforced and concreted.

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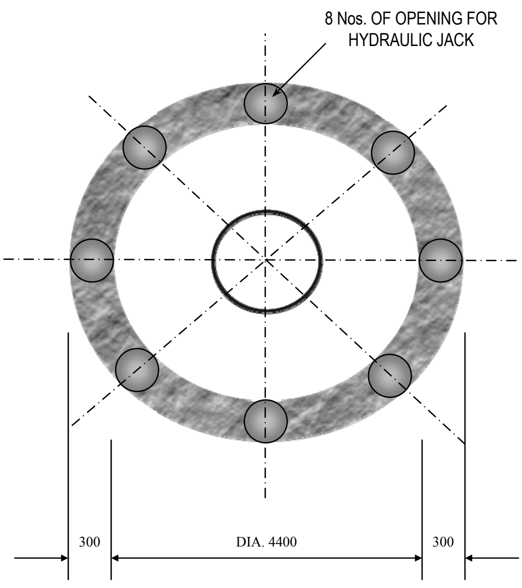

PLAN

Figure 4.21(a). Layout of jacking test.

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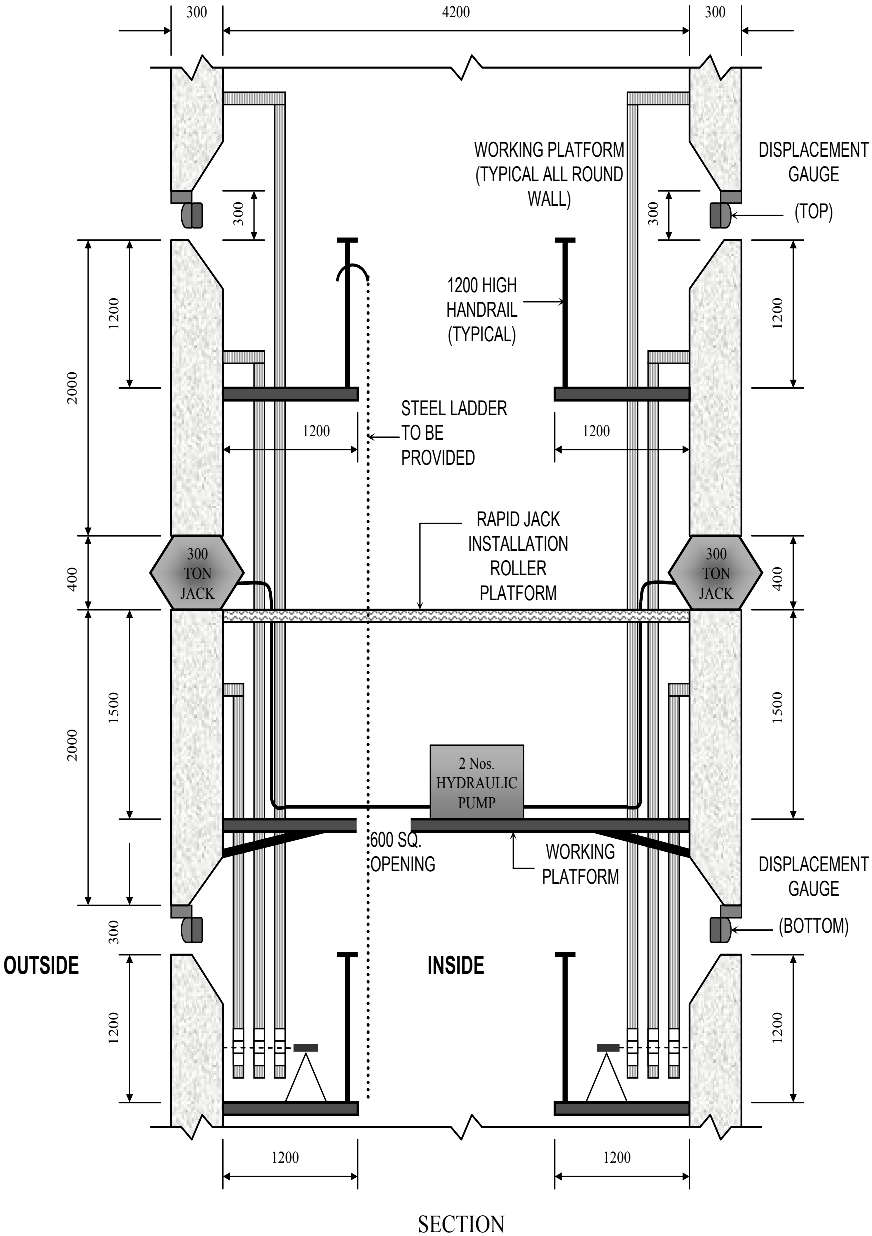

Figure 4.21(b). Jacking test platform.

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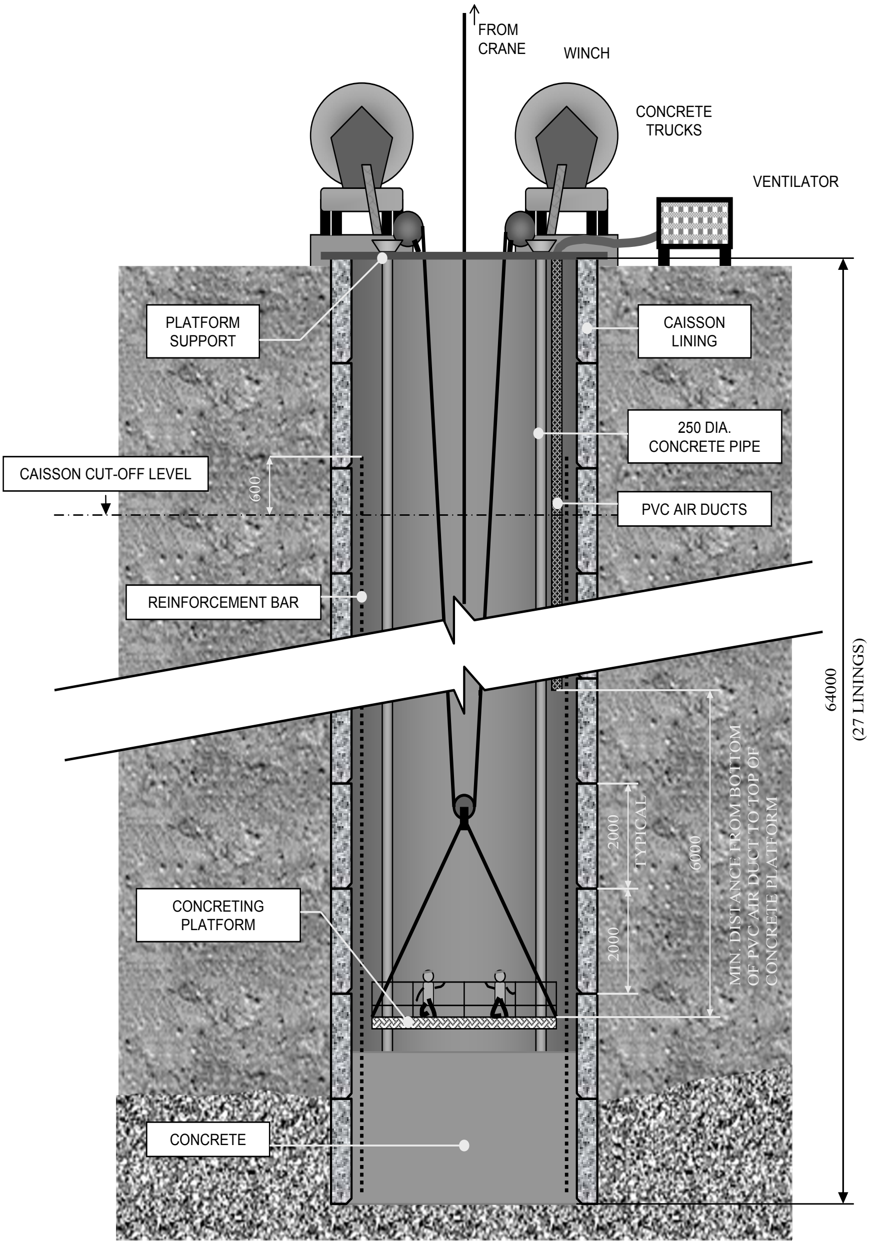


Figure 4.22. Mass concreting of a caisson.

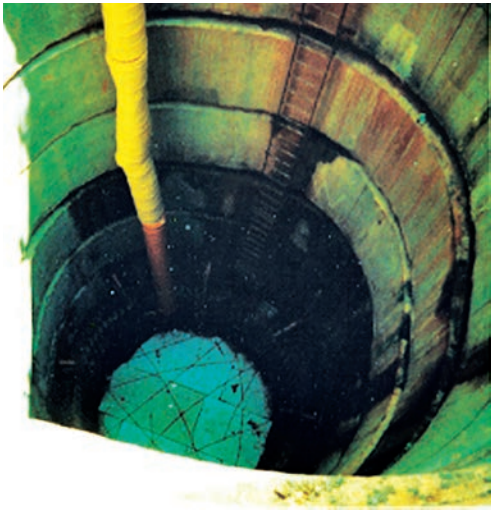
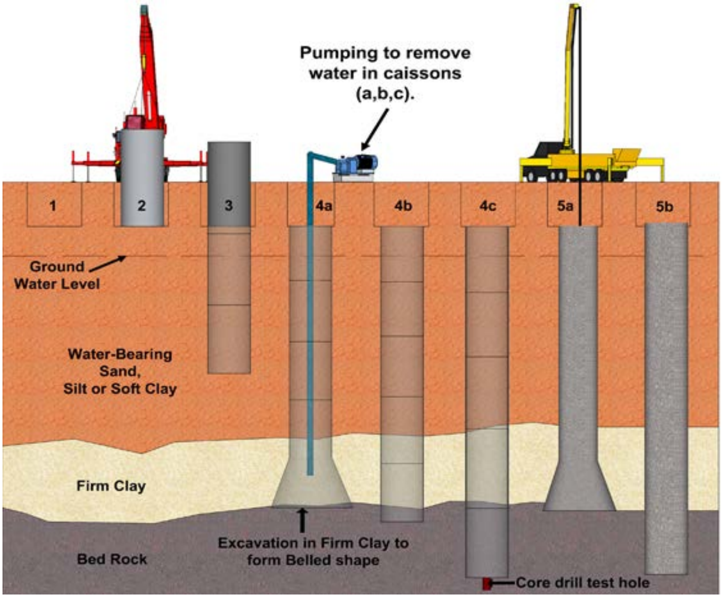


Figure 4.23. Cast caisson to the required height covered with polystyrene for curing.

Figure 4.24. A worker being lowered into the casing of a hand-dug caisson.

Figure 4.25. Concreting through a tremie pipe into a hand-dug caisson.



1. Excavate starting pit to receive caisson.
2. Starting first section of caisson.
3. Advancing through further excavation.

4a. Sealing in clay to form bell.
4b. Sealing in clay to top of bed rock.
4c. Sealing in rock followed by core drill test hole.

5(a & b). Completing caisson by filling with concrete.

Figure 4.26. Construction sequence of gow caissons.

Box Caisson: Box caissons are structures with a closed bottom designed to be sunk into prepared foundations below water level. Box caissons are unsuitable for sites where erosion can undermine the foundations, but they are well suited for founding on a compact inerodible gravel or rock which can be trimmed by dredging (Figure 4.27(a)). They can be founded on an irregular rock surface if all mud or loose material is dredged away and replaced by a blanket of sound crushed rock (Figure 4.27(b)). Where the depth of soft material is too deep for dredging they can be founded on a piled raft (Figure 4.27(c)).

After the box caisson is sunk to the required location, crushed rocks or sand is backfilled into the box to act as an additional weight on the bearing stratum. This is to ensure the rigidity of the foundation for the structure. An example of the use of box caisson is the Kepple-Brani Road Link as shown in Figure 4.28. Figure 4.29(a) shows the construction of a box caisson in a floating dock. Figure 4.29(b) shows the tug boat in operation. Figure 4.29(c) shows the filling of sand into a caisson. Other examples of the use of box caissons include the Jurong Island Linkway and the Causeway to Johore Bahru. Figure 4.29(d) shows the filled caissons forming the base of a road link.

Pneumatic Caisson: A concrete box built with an airtight chamber at the bottom is constructed on ground. The air is compressed balancing with the groundwater pressure to prevent the groundwater from getting into the box. As soil is excavated and removed, the box is gradually sunk into the ground. Steel shafts are connected to the pressurised working chamber as access for workers and excavation machinery. The shafts are equipped with locks to regulate the difference between the atmospheric pressure on the ground and the pressure in the chamber. Excavate 4 m downwards, construct the caisson lining and sink further. Repeat the process to the desire depth. At the designed depth, test for soil bearing capacity, remove the equipment from the working chamber and fill with concrete. An example of the method can be found in Oriental Shiraishi Corporation homepage (http://www.orsc.co.jp).

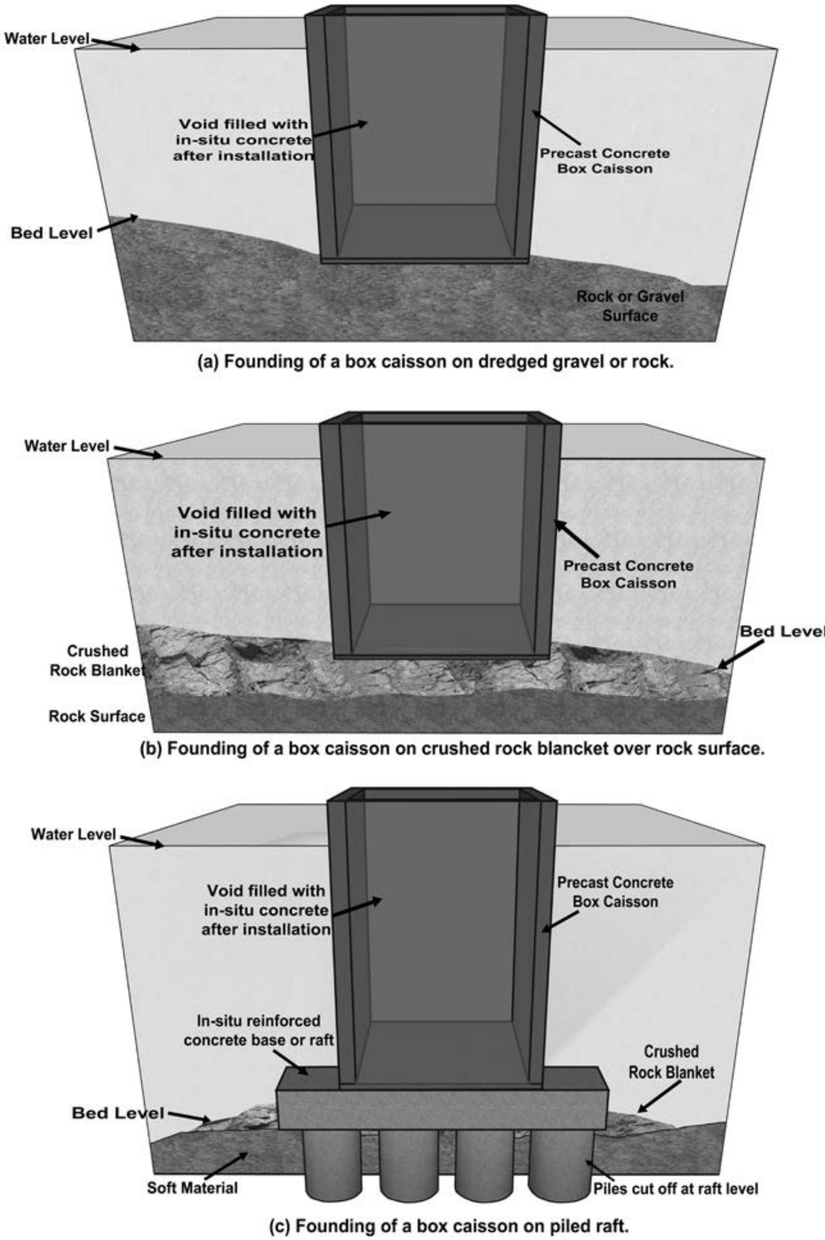


Figure 4.27. (a) Founding of a box caisson on dredged gravel or rock, (b) founding of a box caisson on crushed rock blanket over rock surface, (c) founding of a box caisson on piled raft.

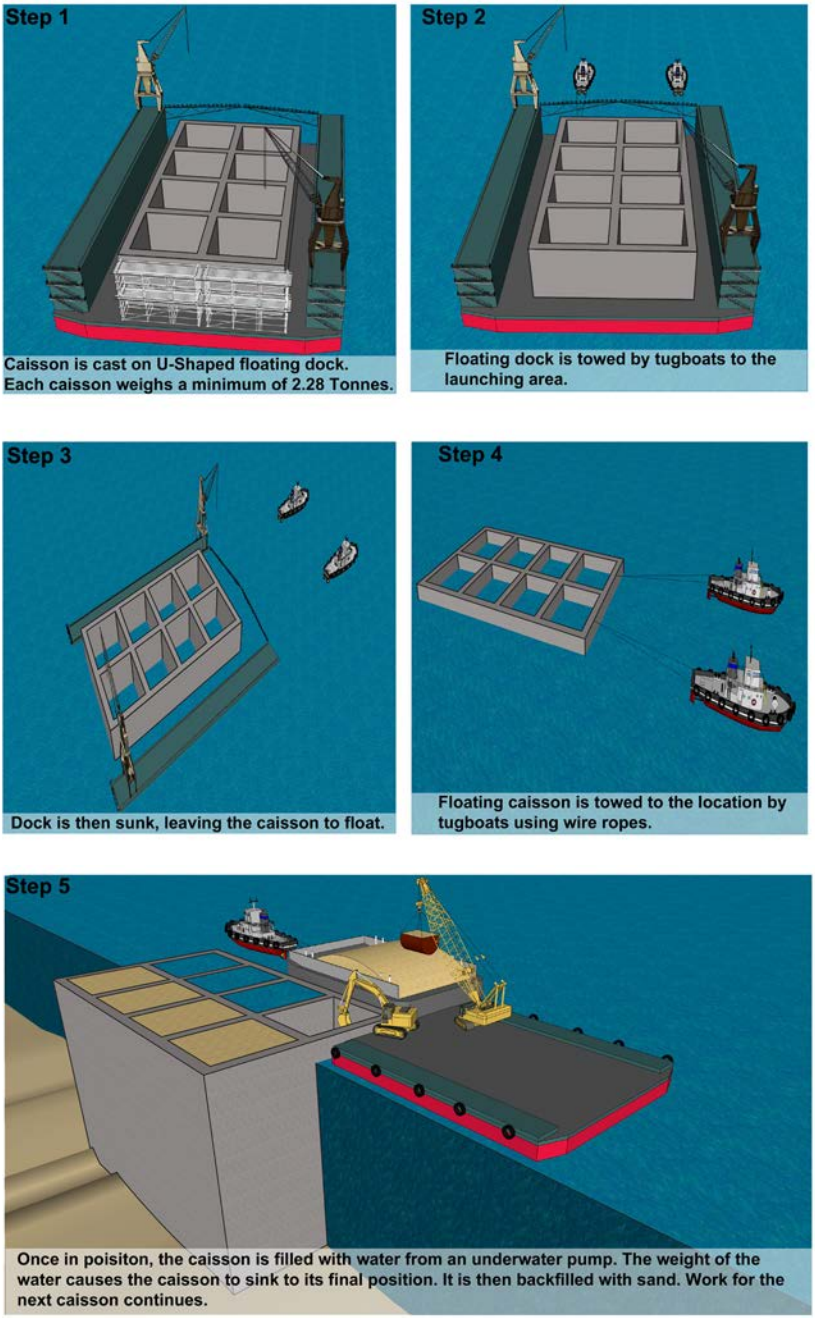


Figure 4.28. Typical construction sequence for a causeway using caissons.

4.8. Piles

A pile can be loosely defined as a column inserted in the ground to transmit the structural loads to a lower level of subsoil. Piles have been used in this context for hundreds of years and until the twentieth century

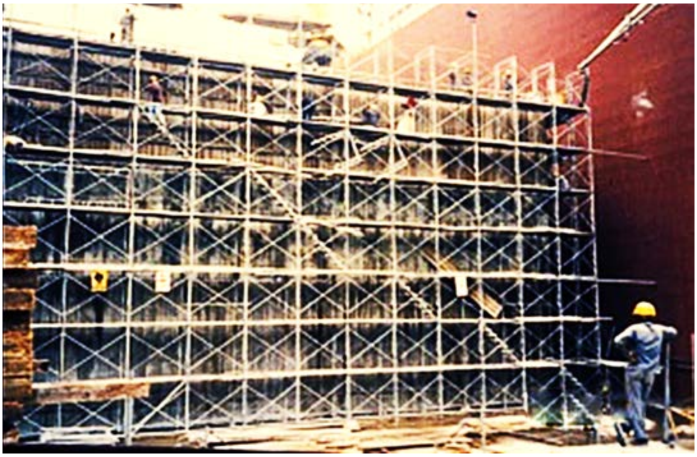


Figure 4.29(a). Construction of a box caisson in a floating dock by the yard. Each caisson is 6 m × 6 m × 12 m.

Figure 4.29(b). A tug boat in operation and a submerged caisson filled with water.

Figure 4.29(c). Filling of sand into a caisson.

Figure 4.29(d). Caissons filled with sand forming the base of a road link.

were at large of driven timber. Today, a wide variety of materials and methods are available to solve most of the problems encountered when confronted with the need for deep foundation.

The construction process of piles can be broadly characterised by the installation and testing. However, there are many proprietary types of piles and the installing process for each type differed.

Piles may be classified by the way there are formed i.e. *displacement piles* and *non-displacement piles*.

The classification of displacement and non-displacement piles is shown in Figure 4.30. The displacement in the soil is the pressure that the pile exerts on the soil as a result of being driven into the soil. In deciding upon the type of piles to use for a particular construction, the following should be considered:

- Superstructure design and the site area.
- Soil conditions and surrounding buildings and structures (e.g. underground tunnels).
- Availability of equipment and site constraints.
- Knowledge of the pros and cons of various piling systems.

Piles may be classified as either end-bearing or friction piles, according to the manner in which the pile loads are resisted.

- End bearing: The shafts of the piles act as columns carrying the loads through the overlaying weak subsoils to firm strata into which the pile toe has penetrated. This can be a rock strata or a layer of firm sand or gravel which has been compacted by the displacement and vibration encountered during the driving.
- Friction: Any foundation imposes on the ground a pressure which spreads out to form a pressure bulb. If a suitable load bearing strata cannot be found at an acceptable level, particularly in stiff clay soils, it is possible to use a pile to carry this pressure bulb to a lower level where a higher bearing capacity is found. The friction or floating pile is mainly supported by the adhesion or friction action of the soil around the perimeter of the pile shaft.

However, in actual practice, virtually all piles are supported by a combination of skin friction and end bearing [26–34].

4.8.1. *Non-Displacement Pile*

4.8.1.1. *Bored Pile*

Sometimes referred to as replacement piles but more commonly as bored piles. They are formed by boring/removing a column of soil and replaced with steel reinforcement and wet concrete cast through

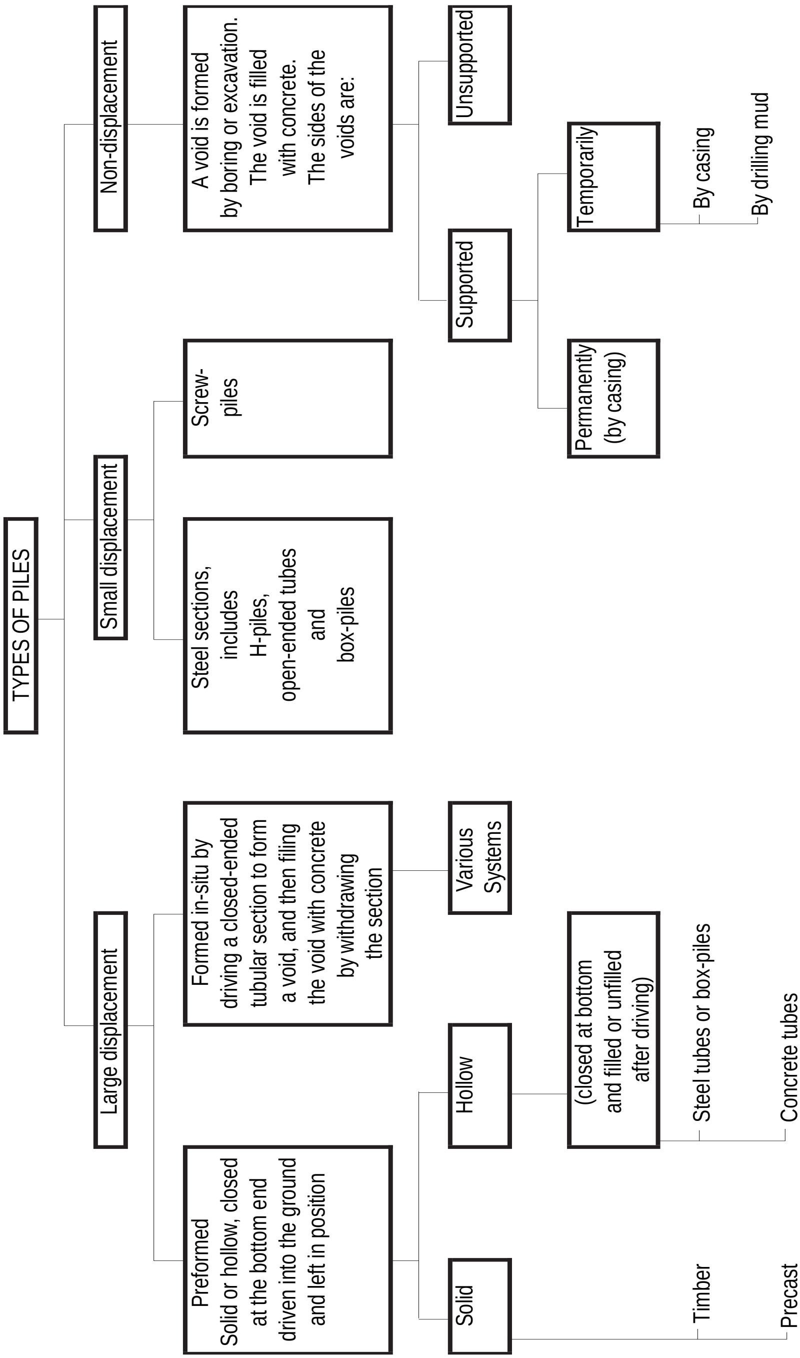


Figure 4.30. The classification of displacement and non-displacement piles.

a funnel or tremie pipe. For soft grounds and where the water table is high, bentonite may be used during boring to resist the excavation and water inflow before casting. Bored piles are considered for sites where piling is being carried out in close proximity to existing buildings where vibration, dust and noise need to be minimised. They are also used instead of displacement piles in soils where negative skin friction is a problem. Negative skin friction is the soil resistance acting downward along the pile shaft as a result of downdrag and inducing compression in the pile. Bored piles of diameter ranges from 100 mm (micro-piles) to 2.6 m are common.

The construction sequence of a typical bored pile is shown in Figure 4.31:

- Set out and peg the exact location of the piles.
- With the boring rig fitted with an augering bit, bore the initial hole for the insertion of the temporary casing.
- Place the casing using a vibratory hammer with the top slightly higher than the ground level (Figure 4.32). The casing serves to align the drilling process as well as to prevent the collapse of the soil from the ground surface.
- Proceed with boring using an augering bit (Figures 4.33 and 4.34). The auger can be of Cheshire or helix auger which has 1½ to two helix turns at the cutting end (Figure 4.35). The soil is cut by the auger, raised to the surface and spun off the helix to the side of the borehole. Alternatively, a continuous or flight auger can be used where the spiral motion brings the spoil to the surface for removal.
- The base or toe of the pile can be enlarged or underreamed up to three times the shaft diameter to increase the bearing capacity of the pile (Figure 4.36).
- For deeper boreholes (> 20 m), a boring bucket may be used (Figure 4.37(a)). The boring bucket is designed with a shuttle to drill into the soil and carry the soil to the surface without it being washed away in the process. On the surface, the shuttle is opened to drop the soil into a soil pit (Figure 4.37(b)).
- Bentonite is commonly used to resist the excavation and water seepage. Bentonite mix ratio of 1:20 is common (See Chapter 5).

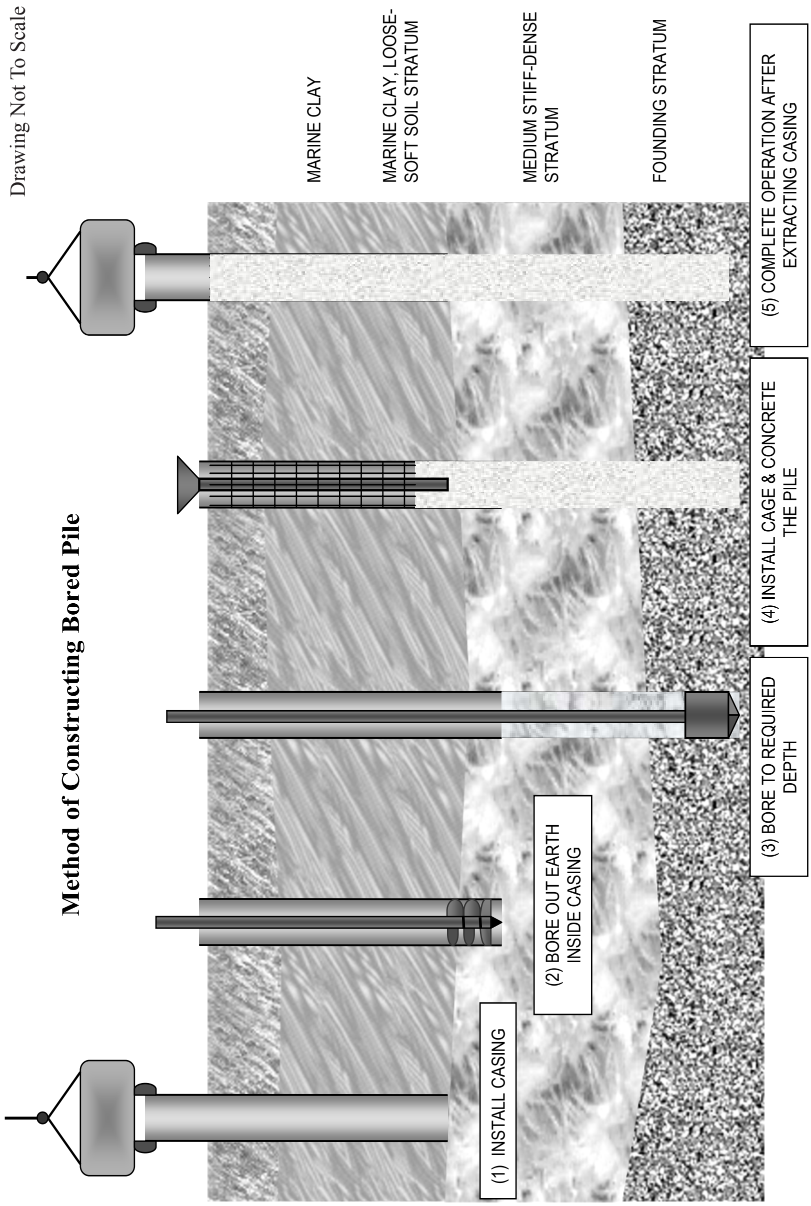


Figure 4.31. An example of bored pile construction.

Figure 4.32(a). Insertion of steel casing using vibro hammer operated by the crane operator.

Figure 4.32(b). Protruded steel casing for safety reasons.

Figure 4.33(a). Augering bits of different diameters.

Figure 4.33(b). An augering bit with industrial diamond "teeth" for cutting rock.

- If boulders or intermediate rock layers are encountered, a core bit (Figure 4.38), a chisel drop hammer (Figure 4.39), or cold explosive may be used to break up the rocks.
- Lighter debris is displaced by the bentonite. A cleaning bucket is used to clear the crush rocks and flatten the surface at the bottom (Figure 4.40).
- At the rock strata, a reverse circulation rig (RCD) may be used to bore into the rock. Piles socketed into hard rock with penetration ranging from 800 mm to 1.6 m is common.

Figure 4.34(a). Removal of soils using an auger.

Figure 4.34(b). Augered soil revealing marine clay.

- Reverse circulation drilling is common for mineral exploration works. It offers a cheaper option to a good quality sampling that nearly equals that of diamond coring. The reverse circulation rig is equipped with tungsten carbide percussion drill bits/teeth (Figure 4.41). Air tubes are inserted near the drilling to circulate the drilled rocks in the bentonite slurry which are then pumped out for filtering/processing.

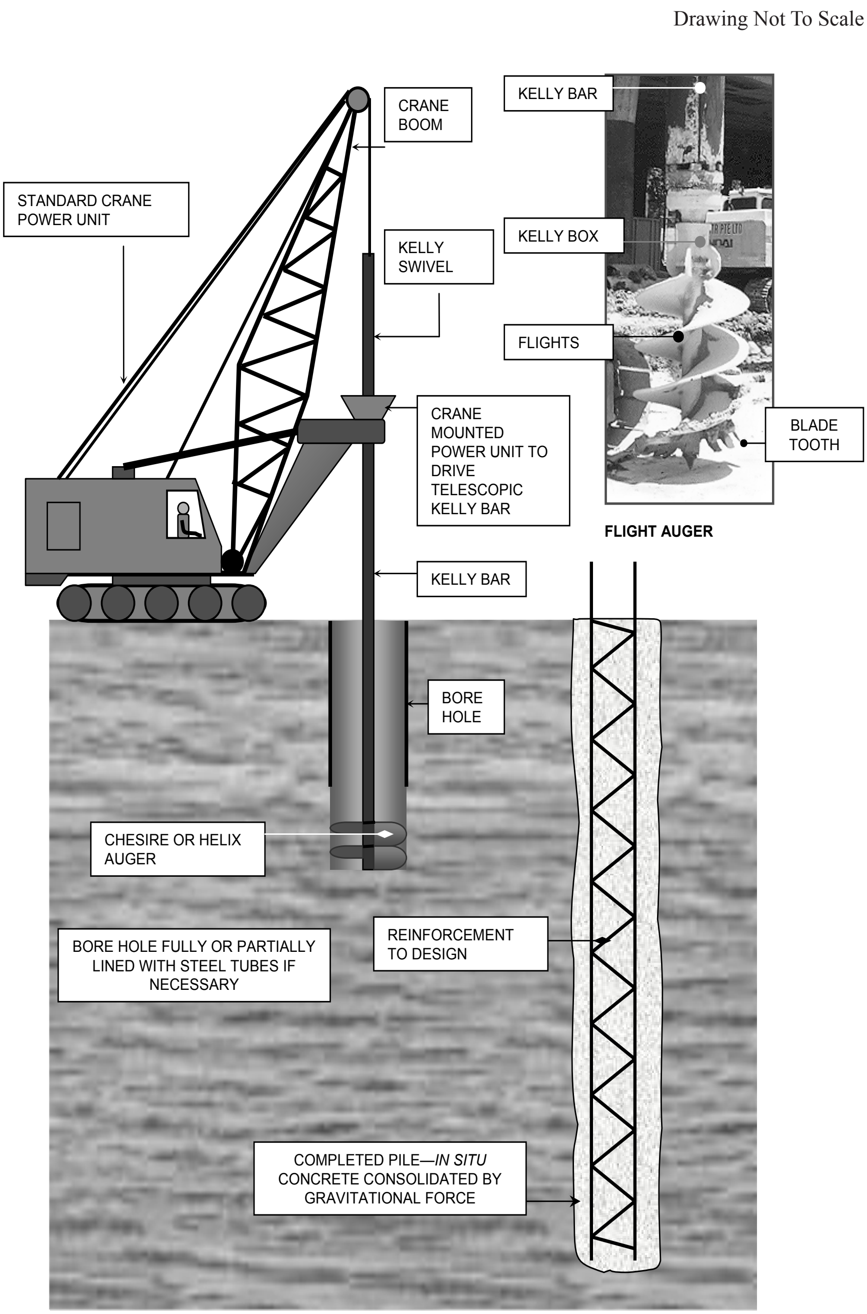


Figure 4.35. Typical rotary bored pile details.

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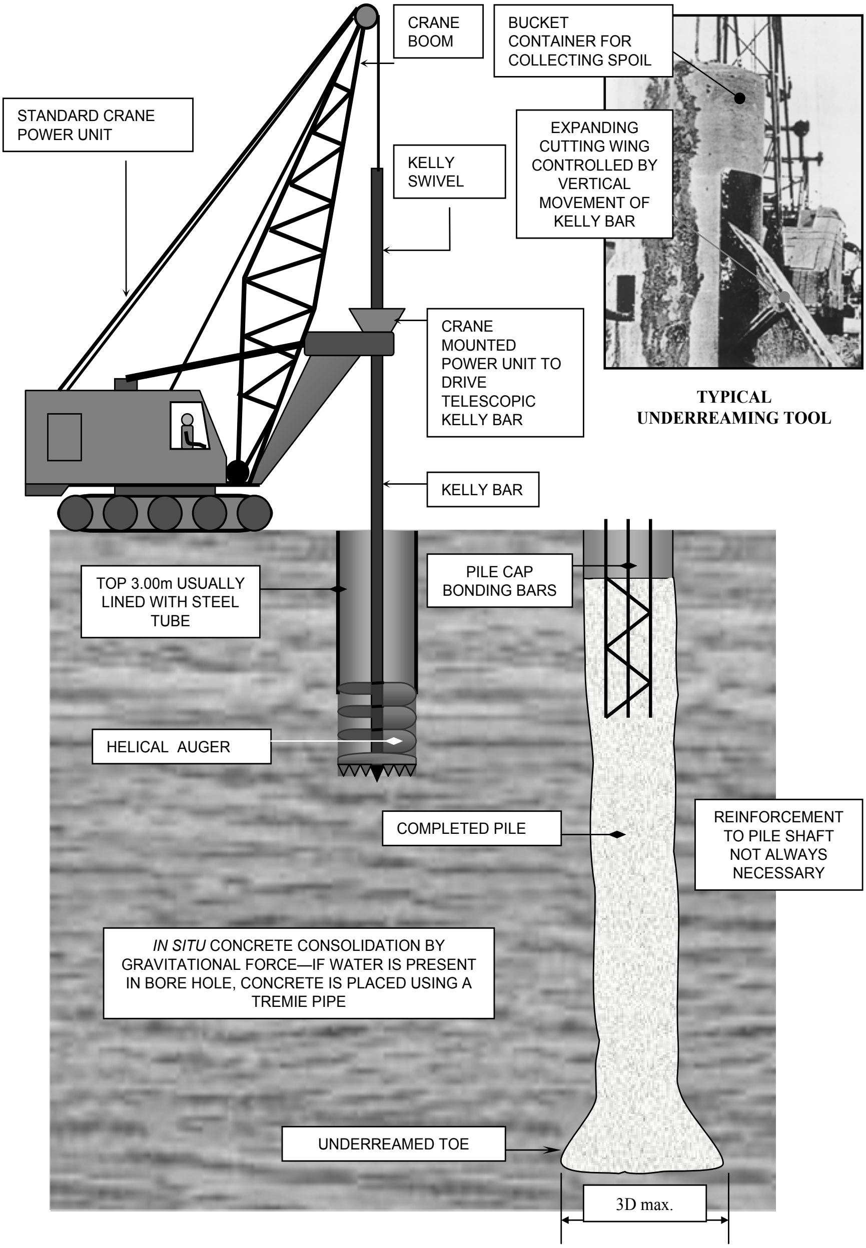

Figure 4.36. Typical large diameter bored pile details.

Figure 4.37(a). A boring bucket.

Figure 4.37(b). Shuttle opened to dump waste onto a soil pit.

Figure 4.38. A boring core bit for the boring process.

Figure 4.39. Chisels used to break up boulders.

Figure 4.40. A cleaning bucket to clear the crush rocks and flatten the surface at the bottom of a bore hole.

Figure 4.41. Drilling bits of the reverse circulation rig (RCD).

- Before the insertion of the reinforcement cage and concrete casting, the high-density contaminated bentonite needs to be changed or it would mix and weaken the concrete. The high-density contaminated bentonite being heavier is pumped out from the bottom of the borehole and fresh bentonite pumped in from the top. An air pipe is inserted near the bottom to stir and circulate the slurry.
- The fresh, low-density bentonite is then tested for purity. Common tests include the sand content test, viscosity test, alkalinity (pH) test.
- Insert reinforcement cage with proper spacers (Figure 4.42).
- Pour concrete with the use of a hopper and tremie pipes (Figure 4.43) to about 1 m higher than the required depth. The excess concrete contains contaminants displaced by the denser concrete and will be hacked off.
- Remove steel casing (Figure 4.44) after concreting.
- Excavate to cut-off level. Hack off excess contaminated concrete on the top exposing the reinforcement. Pour lean concrete around the bored pile. Install formwork for the pile cap and cast concrete (Figure 4.45).

Figure 4.42(a). Insertion of reinforcement cage. Note the 1.2 m lapping between one cage section to another.

Figure 4.42(b). High tensile steel rods are tied to circular rings attached with concrete spacer.

Advantages:

(a) Length can readily be varied to suit the level of bearing stratum.
(b) Soil or rock removed during boring can be analysed for comparison with site investigation data.
(c) *In situ* loading tests can be made in large diameter pile boreholes, or penetration tests made in small boreholes.
(d) Very large (up to 6 m diameter) bases can be formed in favourable ground.
(e) Drilling tools can break up boulders or other obstructions which cannot be penetrated by any form of displacement pile.
(f) Material forming pile is not governed by handling or driving stresses.
(g) Can be installed in very long lengths.
(h) Can be installed without appreciable noise or vibration.
(i) No ground heave.
(j) Can be installed in conditions of low headroom.

Disadvantages:

(a) Concrete in shaft susceptible to squeezing or necking in soft soils where conventional types are used.

Figure 4.43(a). A hopper to be connected to the top section of the tremie pipe. The hopper acts as a funnel to facilitate the concrete pouring.

Figure 4.43(b). Sections of tremie pipes to be connected to the length required.

Figure 4.43(c). Tremie pipe hoisted up and down to provide some vibration.

Figure 4.44. The removal of casing after concreting.

Figure 4.45(a). Excavate to the cut-off level.

Figure 4.45(b). Hack off the top contaminated excess concrete and expose the reinforcement.

Figure 4.45(c). Pour lean concrete around the bored pile.

Figure 4.45(d). Construct formwork and cast the pile cap.

(b) Special techniques needed for concreting in water-bearing soils.
(c) Concrete cannot be readily inspected after installation.
(d) Drilling a number of piles in group may cause loss of ground and settlement of adjacent structures.

4.8.1.2. *Barrette Pile*

Barrettes are rectangular reinforced concrete piles that are orientated to accommodate high horizontal forces and moments in addition to vertical loads. Examples that have incorporated barrette piles into the foundation systems are the Petronas Twin Towers, the Capital Tower, the expansion of Adam Road/Farrer Road Interchange, the Sail@Marina Bay (Figure 4.46), the new Serangoon mass transit railway station and the new SGX Centre [35].

The method and equipment used in the construction of barrette piles is very similar to that of a diaphragm wall (Chapter 5). Barrette piles have been used for very deep foundations, as in the case of the Petronas Twin Towers, the use of 208 barrettes with a depth from 40 m to 125 m was reported [35].

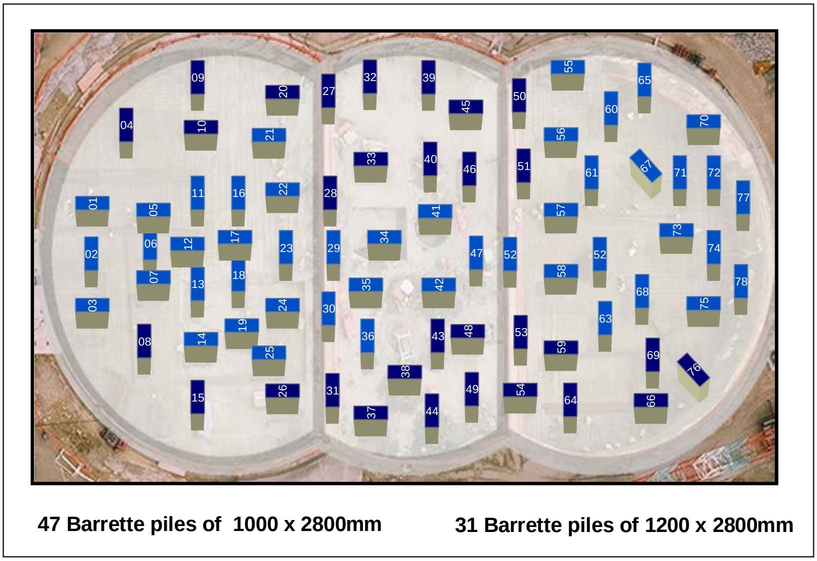

Figure 4.46(a). An example of site layout of oriented barrette piles.

Figure 4.46(b). Example of a barrette pile.

Due to its rectangular shape, a barrette pile has several advantages over bored piles of the same section:

- Offers higher resistance to horizontal stress and bending moments.
- Enhances adjustment to structures, so that one single pile is sufficient under each column or bearing unit.
- Provides higher lateral friction than a circular pile of the same section due to its larger perimeter.

4.8.2. *Displacement Piles*

Displacement piles refer to piles that are driven, thus displacing the soil, and include those piles that are preformed, partially preformed or cast in place. This is the most cost efficient piling method but may not be suitable for areas sensitive to noise, vibration and dust. The presence of boulders can also hinder the use of driving piles.

Advantages:

(a) Material forming piles can be inspected for quality and soundness before driving.
(b) Not susceptible to squeezing or necking.
(c) The pile's carrying capacity can be monitored or 'felt' during the piling process.
(d) The construction operation is not affected by groundwater.

(e) Projection above ground level advantageous to marine structures.
(f) Can be driven to long lengths.
(g) Can be designed to withstand high bending and tensile stresses.

Disadvantages:

(a) May break during driving, necessitating replacement piles.
(b) May suffer unseen damage which may reduce carrying capacity.
(c) Uneconomical if cross-section is governed by stresses due to handling and driving rather than compressive, tensile or bending stresses caused by working conditions.
(d) Noise, vibration and dust levels due to driving may be unacceptable.
(e) Displacement of soil during driving may lift adjacent piles or damage adjacent structures.
(f) Not suitable for situations with low headroom.

4.8.2.1. *Precast Reinforced Concrete Piles*

Come in different sizes and lengths, they are driven by drop hammers or vibrators using a piling rig as shown in Figure 4.47(a). They provide high strength and resistance to decay. They are however heavy, and because of its brittleness and low tensile strength, cares in handling and driving is required. Cutting requires the use of pneumatic hammers, cutting torches, etc. The construction sequence of a typical precast reinforced concrete pile is as follows:

- Set out the position of each pile and to establish the temporary benchmarks (TMB) on site for the determination of the cut-off levels of piles.
- Check the verticality of the leader of the piling rig using a plumb or spirit level.
- Provide markings along the pile section to enhance recording of penetration and to serve as a rough guide to estimate the set during driving.
- Install mild steel helmet (Figure 4.47(b)). Protect pile head/joint plate (Figure 4.48) with packing or cushioning within e.g. a 25 mm thick plywood between the pile head and the helmet.

- Hoist up and place the pile in position (Figure 4.49).
- Check on verticality regularly (Figure 4.50).
- Proceed with the hammering. Monitor pile penetration according to the markings on the pile. When the rate of penetration is low, monitor pile penetration over 10 blows (Figure 4.51). Hold one end of a pencil supported firmly on a timber board not touching the pile. The other end of the pencil marks the pile displacements on a graph paper adhered on the pile over 10 blows.

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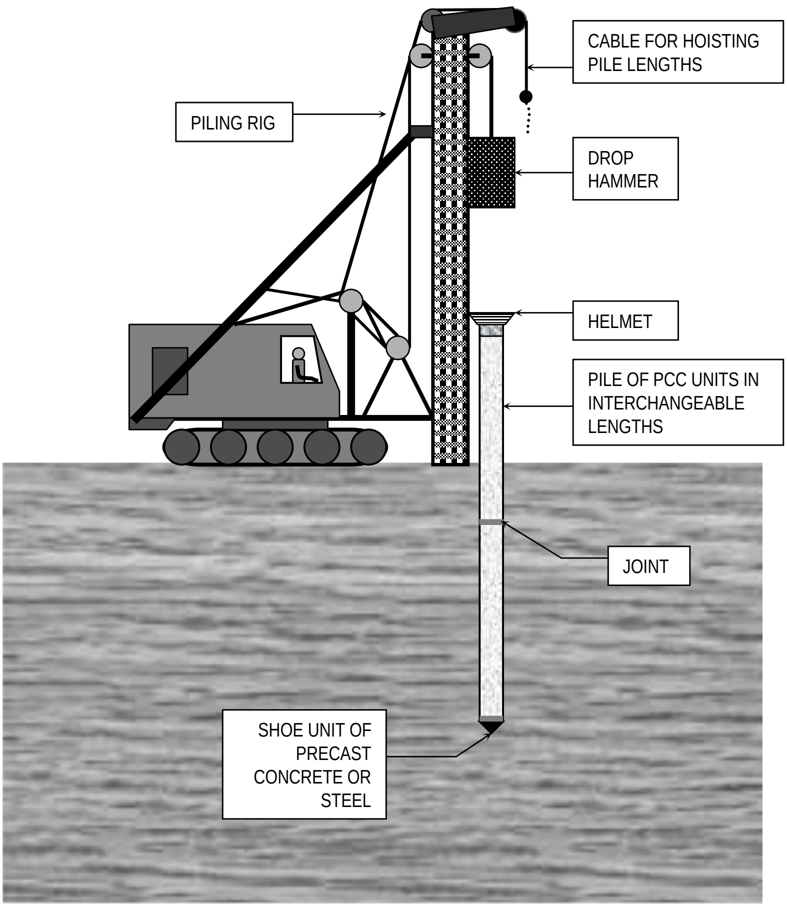

Figure 4.47(a). Precast reinforced concrete piles driven by a drop hammer.

Figure 4.47(b). A mild steel helmet with a 25 mm thick plywood between the helmet and the pile head.

- Stop piling if the displacement is less than the designed displacement over 10 blows. Otherwise, continue with the piling process.
- Lengthening of pile can be done by means of a mild steel splice sleeve and a dowel inserted in and drilled through the centre of the pile. A splice sleeve is a mechanical coupler for splicing reinforcing bars in precast concrete. Reinforcing bars to be spliced are inserted halfway into the cylindrical steel sleeve. The connection is sealed with grout or epoxy resin. It can also be done by welding the pile head/joint plate which were pre-attached to both ends of a pile in the manufacturing process (Figures 4.52 and 4.53).

4.8.2.2. *Steel Preformed Piles*

H-piles or universal steel beam in the form of wide-flange is commonly used (Figure 4.54). They do not cause large displacement and is useful where upheaval of the surrounding ground is a problem. They are capable of supporting heavy loads, can be easily cut and can be driven to great depth. The driving method for steel piles is similar to that of precast reinforced concrete piles. The handling and lifting of a steel pile is less critical due to its high tensile strength. Lengthening of steel piles is through welding (Figure 4.55). Care must be taken in the welding of joints to ensure that they are capable of withstanding driving stresses without failure. A protective steel guard should be welded at the joints when necessary.

Figure 4.48(a). Pile heads/jointing plates inserted.

Figure 4.48(b). Pile heads with dowels (male and female).

Figure 4.49(a). Hoist up the pile and insert the helmet.

Figure 4.49(b). Place pile in position into the piling rig.

4.8.2.3. *Composite Piles*

Also referred to as partially preformed piles, composite piles combine the use of precast and *in situ* concrete and/or steel. They are an alternative to bored and preformed piles for sites with the presence of running water or very loose soils. There are many commercial proprietary systems available. The common generic types are the shell piles and cased piles (Figures 4.56 and 4.57) [36].

4.8.2.4. *Driven In Situ/Cast-in-Place Piles*

The pile shaft is formed by using a steel tube which is either top driven (Figure 4.58) or driven by means of an internal drop hammer working on a plug of dry concrete/gravel as in the case of Franki piles (Figure 4.59). Piles up to 610 mm can be constructed using this method.

Figure 4.50. Guide the verticality using a spirit level and the leader of the rig.

Figure 4.51(a). Marking pile displacements over the first 10 blows.

Figure 4.51(b). Marking pile displacements over the second 10 blows.

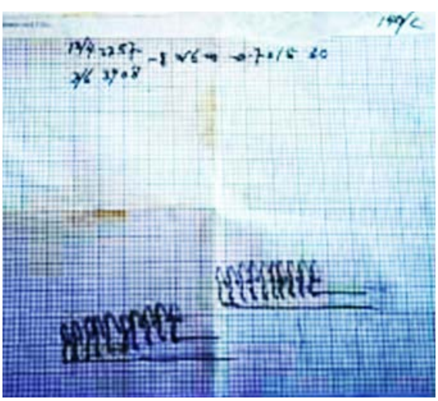

Figure 4.51(c). Markings on the graph paper.

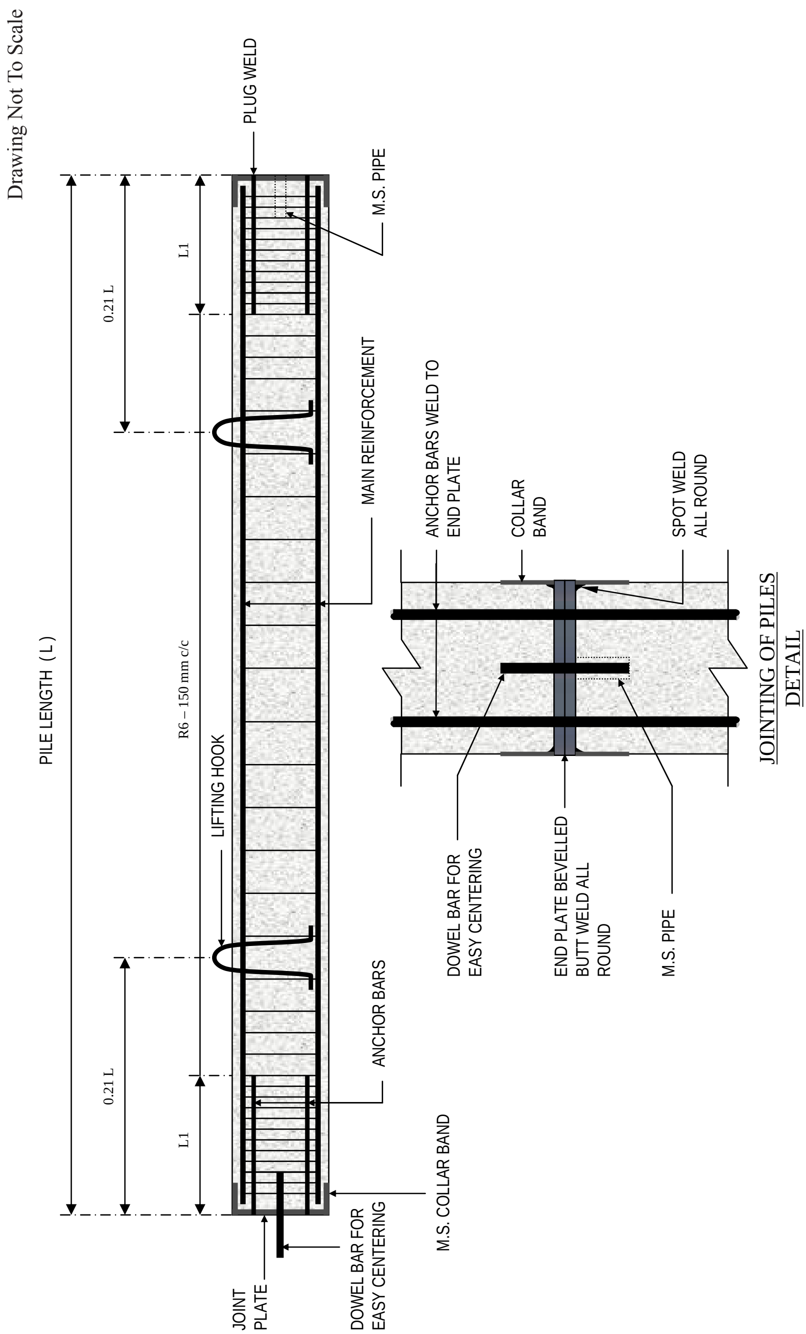

Figure 4.52. Precast reinforced concrete pile and splicing details.

Figure 4.53(a). Lifting of a new pile.

Figure 4.53(b). Insert the new pile to the old.

Figure 4.53(c). Align the new pile using a steel fork.

Figure 4.53(d). Weld the joint plate together.

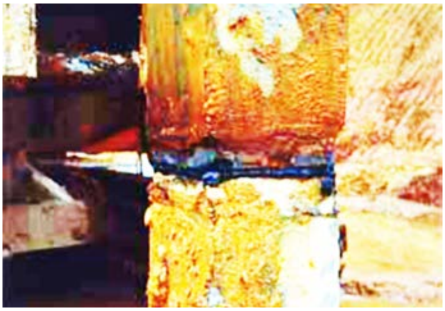

Figure 4.53(e). A welded joint.

Figure 4.54. Steel preformed H-piles.

Figure 4.55. Joint welding of steel piles.

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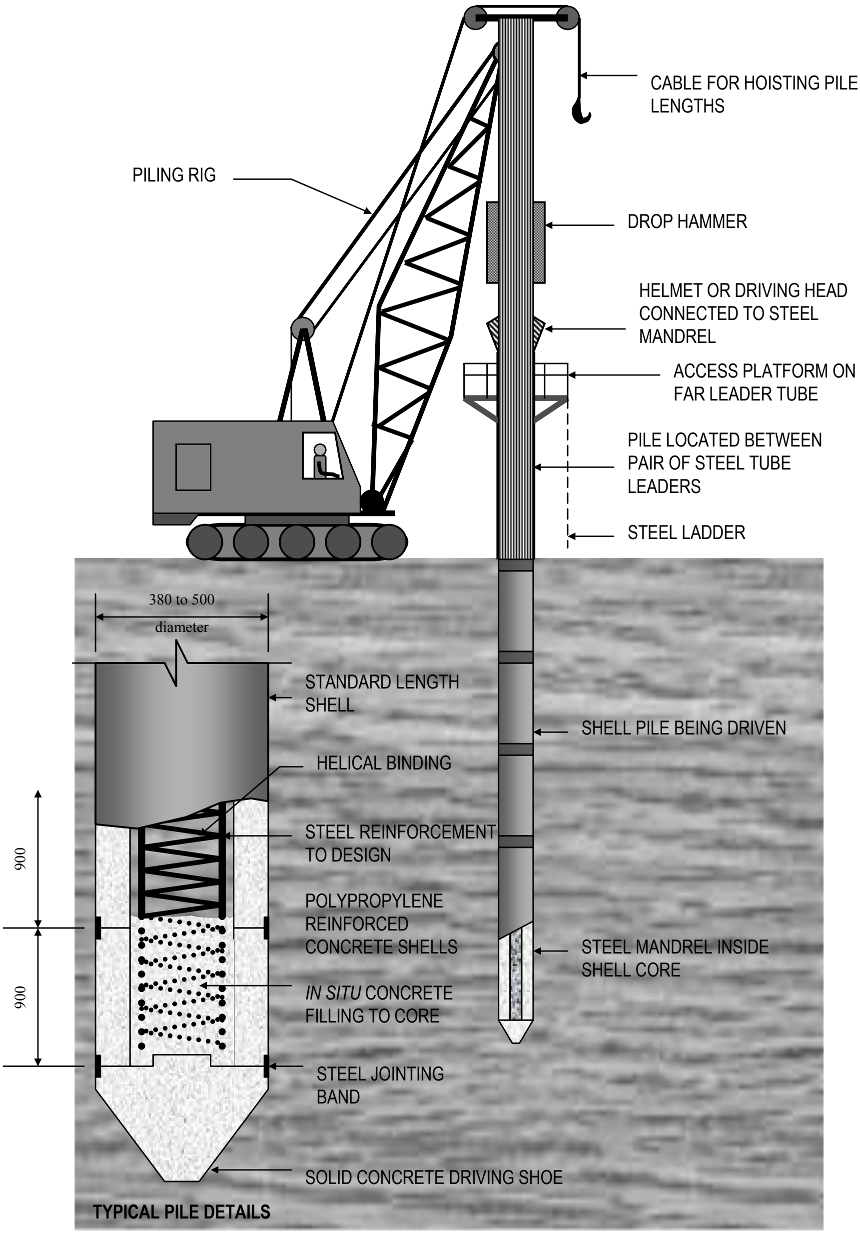

Figure 4.56. A shell pile using precast and *in situ* concrete [36].

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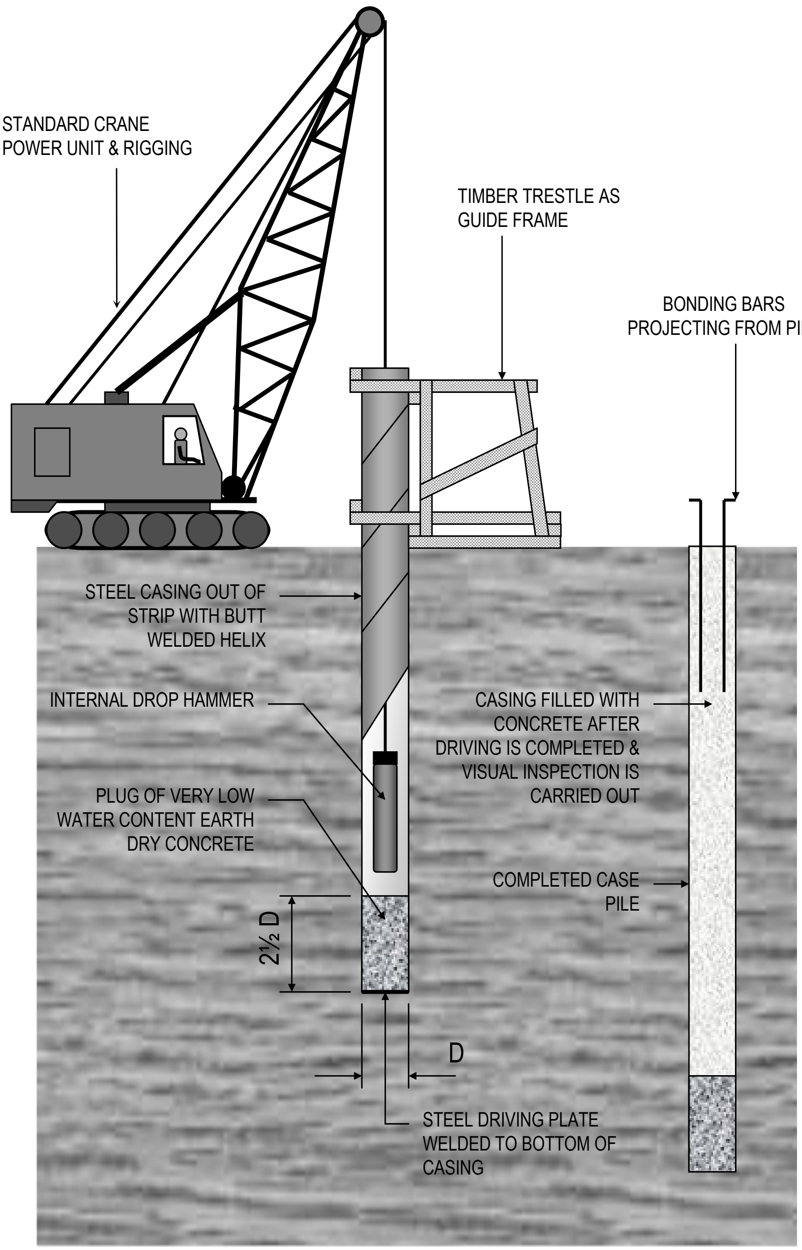

Figure 4.57. A cased pile using steel and *in situ* concrete [36].

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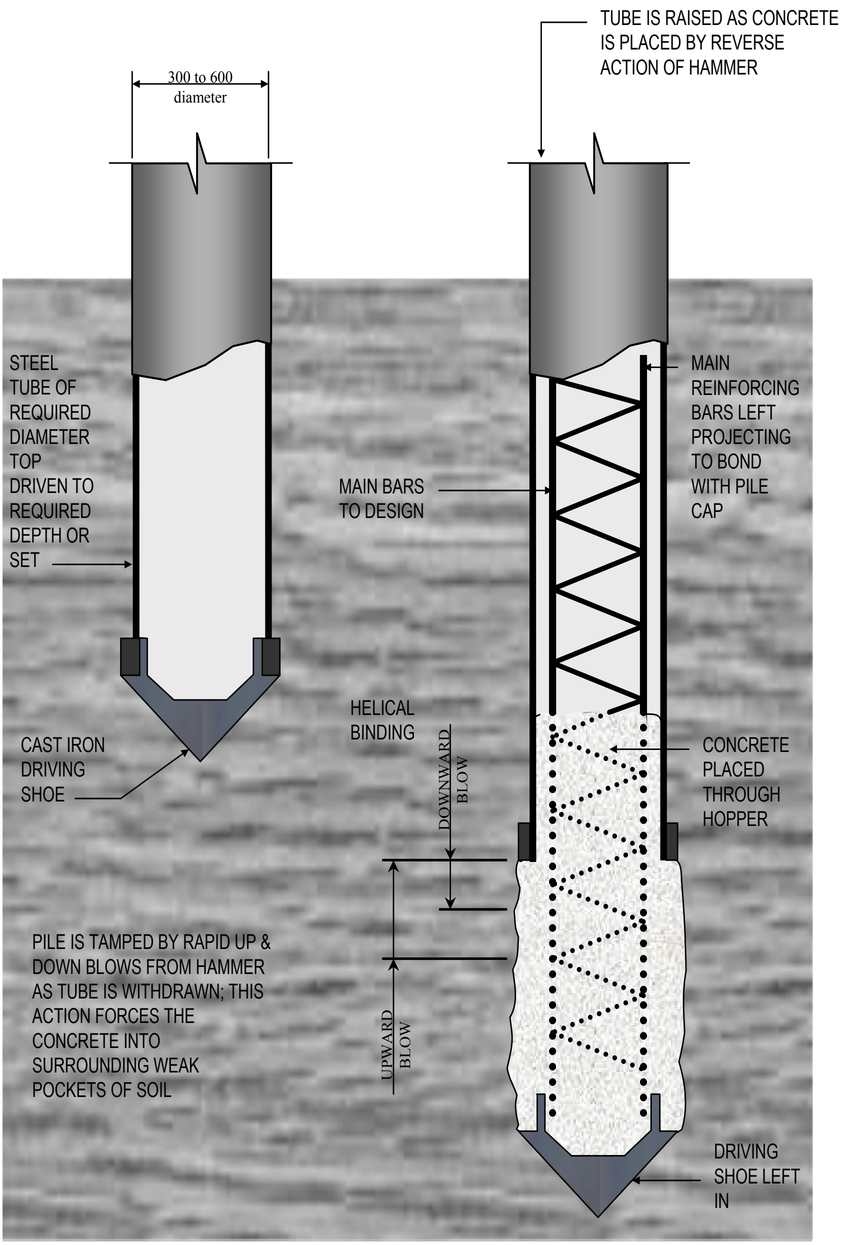

Figure 4.58. Top-driven cast *in situ* piles [36].

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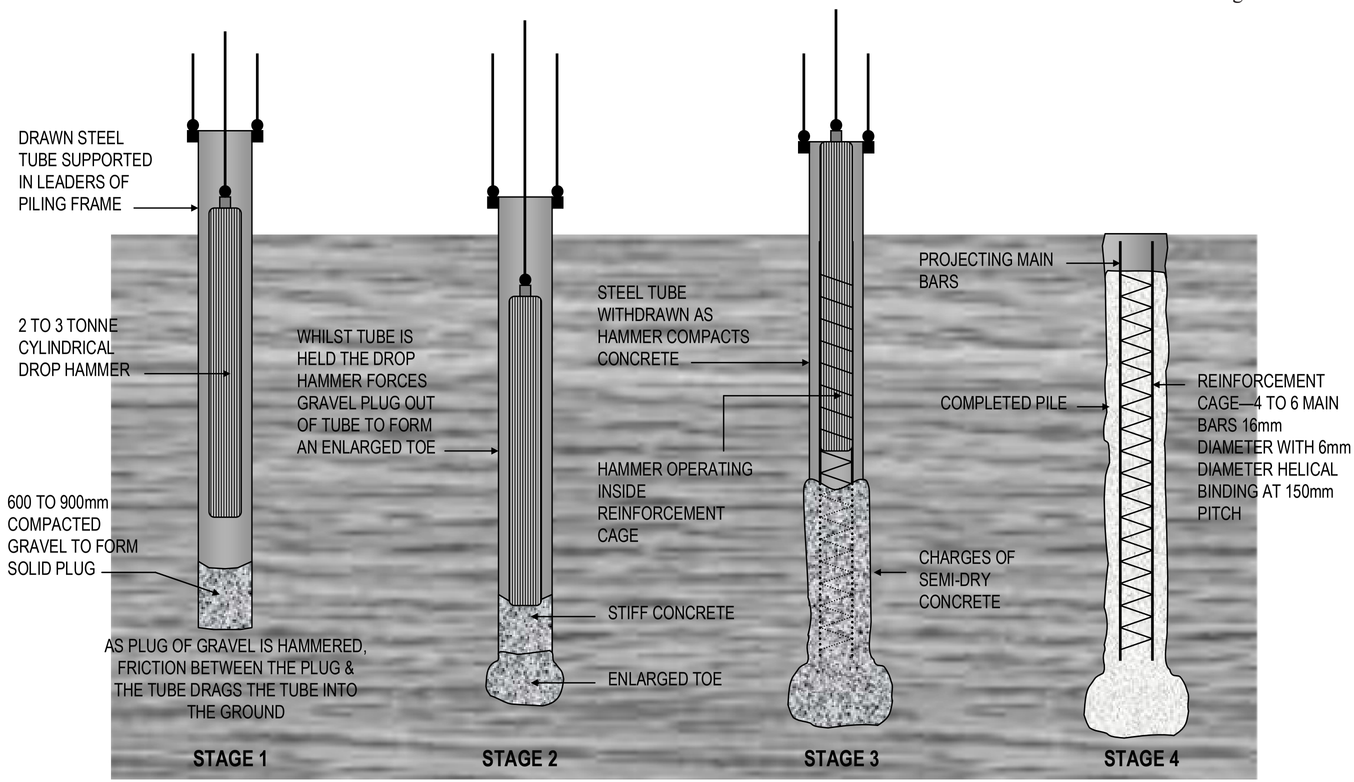

Figure 4.59. Franki driven *in situ* piles [36].

With temporary casing, as the casing is withdrawn during the placing/compacting of concrete, precautions need to be taken when working at depths with groundwater movement to prevent problems associated with necking (narrowing), caused when the groudwater washes away some of the concrete thus reducing the effective diameter of the pile shaft and consequently the concrete cover [36].

4.9. Pile Load Tests

The main objective of forming test piles is to confirm that the design and formation of the chosen pile type is adequate. Pile load tests give information on the performance of the pile, installation problems, lengths, working loads and settlements.

4.9.1. *Static Load Tests*

Static load tests involve the use of a heavy load or a reaction method to counter the application of an axial load to the top of the test pile using one or more hydraulic jacks. The main objectives are to determine the ultimate failure load, capability of supporting a load without excessive or continuous displacement, and to verify that the allowable loads used for the design of a pile are appropriate and that the installation procedure is satisfactory.

Two types of loading are commonly used:

Maintained Load Test — also referred to as working load test, of which load is increased at fixed increments up to 1.5 to 2.5 times its working load. Settlement is recorded with respect to time of each increment. When the rate of settlement reaches the specified rate, the next load increment is added. Once the working load is reached, maintain the load for 12 hours. Thereafter, reverse the load in the same increment and note the recovery.

Ultimate Load Test — the pile is steadily jacked into the ground at a constant rate until failure. The ultimate bearing capacity of the pile is the load at which settlement continues to increase without any further increase of load or the load causing a gross settlement of 10% of the pile diameter. This is only applied to test piles which must not be used as part of the finished foundations but should be formed and tested in such a position that will not interfere with the actual contract but is nevertheless truly representative of site conditions.

4.9.1.1. *Compression Load Test*

As pile foundations are usually designed to carry compression loads transmitted from the superstructure, compression load test is hence the most common test method to assess the load carrying capacity of piles. Compression load test can be conducted using different reaction systems including:

(i) kentledge
(ii) tension piles
(iii) ground anchors
(iv) Osterberg cells.

Criteria for selection include soil conditions, size of the site, number of piles to be tested and haulage costs.

Figure 4.60. Kentledge pile test with stone blocks stacking up on steel channels to act as the weight.

Figure 4.61. Kentledge pile test with a hydraulic jack installed in between the tested pile and the loaded channel. During testing, the hydraulic jack is extended progressively. The applied force versus the displacement of the tested pile is monitored.

(i) Kentledge

Heavy load or kentledge comprises either stone or cast concrete blocks (usually 1 to 2 m^3), pig iron blocks (usually up to 2 tonnes), or any other suitable materials that can be safely stacked up are used to form the reaction weight for the test (Figures 4.60 and 4.61). The weight of the kentledge is borne on steel or concrete cribbings. Main and secondary girders are connected to the pile head in such a way that the load can be distributed evenly. The distance between the test pile and the supporting cribbings should be kept as far as possible (Figure 4.62). The system should be firmly wedged, cleated or bolted together to prevent slipping between members. The centre of gravity of the kentledge should be aligned with that of the test pile to prevent preferential lifting on either side which may lead to toppling. A jack is positioned between the main girder and the test pile.

(ii) Tension piles

A reaction system using tension piles (also called reaction piles or anchors) involves two or more tension piles to form the reaction frame. The outer tension piles are tied across their heads with a steel or concrete beam. The object is to jack down the centre or test pile against the uplift of the outer piles. It is preferable when possible to utilise more than two outer piles to avoid lateral instability and to increase the pull-out resistance. It

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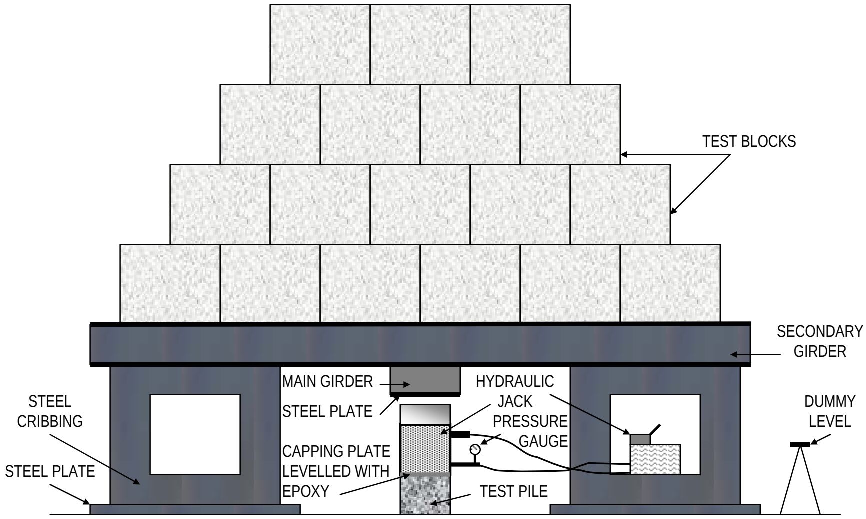

Figure 4.62(a). Front elevation of a typical kentledge set up.

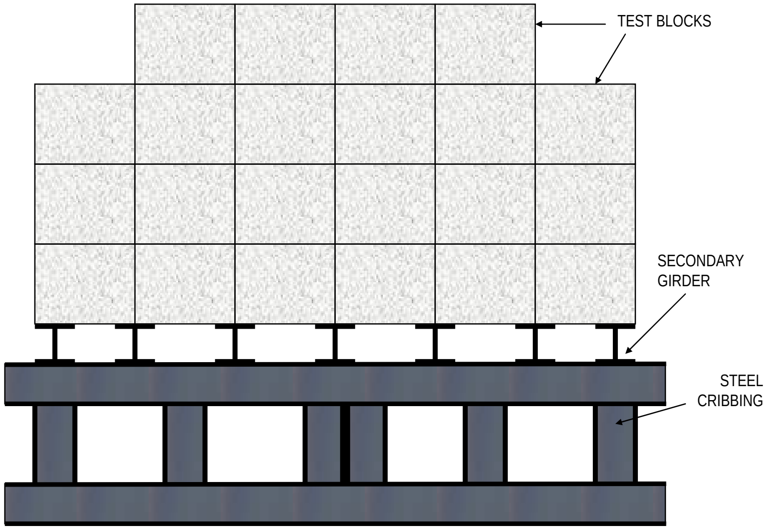

Figure 4.62(b). Side elevation of a typical kentledge set up.

Figure 4.63. An example of pile test using four tension piles (courtesy: Geotechnical Engineering Bureau, State of New York Department of Transportation [37]).

is preferable to employ four, so that a cross-head arrangement can be used at one end of the main beam, enabling the flanges of the cross beam to be bolted to the main loading beam (Figure 4.63).

(iii) Ground anchors

Ground anchors (also called rock anchors) are useful for testing piles which are end bearing on rock. It is achieved using sufficient number of anchor piles to provide adequate reactive capacity and a clear distance from the test pile. The principles are similar to that of tension piles. More on ground anchors is discussed in Chapter 5.

(iv) Osterberg cells

This is an alternative to the conventional static load tests for bored piles where space is a problem as it does not require the use of kentledge, reaction beams and anchor piles. It involves the use of sacrificial hydraulic jack cells (Osterberg-cell or O-cell) cast within the tested pile, with twin

Figure 4.64. Multiple O-cell assembly ready for installation (courtesy: LOADTEST Asia Pte Ltd).

reaction plates similar in diameter to the tested pile at the top and bottom of the cell (Figure 4.64). By incrementally increasing the pressure in the jack, the O-cell expands and works in two directions; upwards against upper skin friction and downwards against base resistance. Movements are measured using strain gauges and reference rods.

4.9.1.2. *Uplift Pile Load Test*

When there is a relative movement between a pile and the soil, shear stress is produced along the interface. The relative movement may be induced by the movement of the pile, or the movement of the soil, or both. When the movement of the pile is downward, the shear stress induced in the pile acts upward, it is called positive skin friction. In the case where the movement of the pile is upward, the shear stress induced in the pile acts downward, it is called negative skin friction [38]. Understanding the skin friction of the soil is important particularly when basement or underground construction is involved. Piles could be uplifted if insufficient skin friction is developed causing the whole underground structure to be uplifted.

Uplift or pull out test on piles is used to determine the negative skin friction of the soil. The test is similar to that of a tension pile test except that the jack pulls the pile upward instead.

4.9.2. *Dynamic Load Tests*

Contrary to static load test which is a direct load test requiring the use of a heavy load or a reaction method, dynamic load test is an indirect method using the wave propagation theory to estimate the condition of a hammer-pile-soil system. The method involves the process of impacting the tested pile with a large drop weight and measuring the compressive stress wave travelling down the pile. Transducers and accelerometers are installed near the top of the pile to measure the reflected wave. Using measurements of strain and acceleration and the principles of wave mechanics, performance such as static pile capacity and pile integrity can be estimated.

High-strain dynamic testing

This method is used for displacement piles to estimate the static axial pile capacity. For displacement piles during the driving process, while the dynamic load is applied to the pile by a pile hammer operating at its normal operating level, strain gauges and accelerometers are installed near the top of the piles and measurement are taken during pile driving. The measurements of strain are converted to force and the measurements of acceleration are converted to velocity. The static pile capacity is estimated using dynamic resistance equations. This test is standardized by ASTM D4945-08 Standard Test Method for High Strain Dynamic Testing of Piles [39].

Low-strain dynamic integrity testing

This is a pile integrity test based on wave propagation theory. It is also known as sonic echo test. As a pile is buried in the ground, quality checks can only be conducted through indirect methods. In this method, a light impact using a hand held hammer is applied to a pile, resulting in a low strain. The impact generates a compression wave that travels at a constant speed down the pile. Anomalies such as inconsistent cross sectional area; voids and cracks produce wave reflections of certain characteristics. An accelerometer placed on top of the pile measures the response to the hammer impact. Given a known stress wave speed, records of velocity at the pile head can be interpreted to reveal pile non-uniformities. This test is standardized by ASTM D5882-07 Standard Test Method for Low Strain Integrity Testing of Deep Foundations [40].

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