PORT WORKS DESIGN MANUAL
PART 3

Guide to Design of Reclamation

Civil Engineering Office
Civil Engineering Department
The Government of the Hong Kong Special Administrative Region

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FOREWORD
(Continuously updated e-version Jun 2023)

This continuously updated e-version of the Port Works Design Manual has incorporated the previously issued Corrigenda No. 1/2018 and No. 1/2020 to facilitate the designers and industry practitioners to carry out coastal design in a more convenient manner.

Practitioners are encouraged to comment at any time to the Civil Engineering Office on the contents of this document, so that improvements can be made to future editions.

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Head, Civil Engineering Office
June 2023

FOREWORD

The Port Works Design Manual presents recommended standards and methodologies for the design of marine works in Hong Kong. It consists of five separate volumes, namely, Part 1 to Part 5. Part 1 mainly covers design considerations and requirements that are generally applicable to various types of marine works. Part 2 to Part 5 are concerned with specific design aspects of individual types of works including piers, dolphins, reclamations, seawalls, breakwaters and beaches. This Manual supersedes the Port Works Manual prepared in the 80's.

This document, Port Works Design Manual: Part 3, gives guidance and recommendations on reclamation design, covering design considerations, stability analysis, settlement assessment and monitoring. It was prepared by a working committee comprising staff of the Civil Engineering Office and Special Duties Office with reference to the latest local and overseas geotechnical and reclamation publications and experiences in consultation with the Geotechnical Engineering Office and other Government departments, engineering practitioners and professional bodies. Many individuals and organizations made very useful comments, which have been taken into account in drafting the document. An independent review was also undertaken by geotechnical experts before the document was finalized. All contributions are gratefully acknowledged.

Practitioners are encouraged to comment at any time to the Civil Engineering Office on the contents of this document, so that improvements can be made to future editions.

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June 2002

Working Committee of Port Works Design Manual : Part 3 **[First published Version in 2002]**

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1. INTRODUCTION

1.1 Purpose

The purpose of the Port Works Design Manual (the Manual) is to offer guidance on the design of marine works and structures normally constructed by the Government of the Hong Kong Special Administrative Region. Such works and structures include public piers, ferry piers, dolphins, reclamations, seawalls, breakwaters, pumphouses, beaches and associated marine facilities. The Manual has been written with reference to the local conditions and experience. Therefore, it may also provide a source of useful data and design reference for marine works and structures constructed by other organizations or parties in Hong Kong.

The Manual is issued in five separate parts. The titles of these parts are:

- Part 1 – General Design Considerations for Marine Works
- Part 2 – Guide to Design of Piers and Dolphins
- Part 3 – Guide to Design of Reclamation
- Part 4 – Guide to Design of Seawalls and Breakwaters

  Part 5 – Guide to Design of Beaches

The recommendations in the Manual are for guidance only and should not be taken as mandatory. Compliance with these recommendations does not confer immunity from relevant statutory and legal requirements. Because of the variable nature of the marine environment, the design of marine works and structures relies particularly on the use of sound engineering judgement and experience. Practitioners should be aware of the limitations of the assumptions employed in a particular theoretical or computational method. Since the marine environment is a field where active research and development are continuing, it is beyond the scope of the Manual to cover all analysis and design methods. Practitioners should be prepared to explore other methods to suit a particular problem and should also realize that many of the methods will continue to evolve.

This part (Part 3) of the Manual gives guidance and recommendations on the design of reclamation, covering such aspects as design considerations, stability analysis, settlement assessment and monitoring of reclamation works. Worked examples are provided in Appendix D to illustrate the application of the recommended design methods. Readers should refer to other parts of the Manual on particular aspects, as necessary.

1.2 Definition, Symbols and References

The definitions of terms and meanings of symbols for the purpose of this part of the Manual are given in the Glossary of Terms and Glossary of Symbols at the end of this document.

The titles of publications referred to in this part of the Manual are listed in the reference section. The reader should consult these original publications for more detailed coverage of particular aspects. For Works Bureau Technical Circulars (WBTC) which are updated regularly, reference should be made to their latest issues.

2. DESIGN CONSIDERATIONS

2.1 General

This chapter gives comments and guidance on the design of reclamation covering such aspects as the reclamation layout, formation level, reclamation methods, fill, treatment of fill, environmental impacts, drainage impacts and so forth.

2.2 Purpose of Reclamation

Reclamation may generally be carried out:

- To provide land for essential major transport infrastructure in order to avoid routing major highways through existing developed areas.
- To provide land for housing, sites for additional community facilities and public open spaces.
- To provide land for port and industrial uses.
- To eliminate areas of badly polluted water and improve hydraulic conditions by
- rearranging the coastline.

The design of reclamation should fulfil the requirements of the planned use and programme for development, particularly with regard to time and settlement characteristics of the reclamation and foundation materials. The timing of future development, location of future roads, drains, buildings and areas of open space are of particular importance. Sometimes it may be necessary to design reclamation for the main initial purpose of beneficial reuse of public fill. In such circumstances, future land use and development is likely to be known only in general terms at the time of reclamation design. It is relatively common for planned use and development of reclamation to change with time, due to changing needs, and account should be taken of this where possible by designing reclamation for flexibility of use.

2.3 Site Investigation

Reclamation projects are usually extensive, and problems during or after the construction, may carry major financial implications. To enable the designer to devise the most appropriate design, it is essential that site investigation is carried out prior to the design of reclamation. In addition to normal geotechnical investigations required for marine works, the investigations may need to cover potential sources of fill materials. In-situ and laboratory testing of soil samples from within the proposed reclamation area should be carried out to determine the strength, settlement and permeability characteristics of the underlying soils. Details of site investigation and soil testing are covered in Geoguide 2 (GEO, 2017a) and Chapter 4 of Part 1 of the Manual.

2.4 Layout

The layout of a reclamation project will largely be governed by town planning considerations.

The major factors restricting the extent of reclamation will be the water depth, the need to maintain fairways, moorings and other marine traffic channels and clearances to the approval of the Director of Marine, and the environmental implications.

The effect of a proposed reclamation on the hydraulic regime must be considered. Investigations should include hydraulic study of the currents, waves and sediment transport, environmental impact assessment and marine traffic impact assessment to ensure that there are no unacceptable effects with respect to:

- Change in normal and extreme wave climates.
- Tidal flushing and water quality.
- Ecology.
- Siltation and seabed scour.
- Shoreline stability of existing beaches.
- Navigation of large and small vessels.
- Operation of piers, wharves and cargo-handling areas.
- Flooding due to tides combined with storm surge.

The effects of reclamation on water flow in general, and currents, waves, storm surge characteristics and sediment transport conditions at and along the shoreline in particular, can cover a far more extensive area than the area immediately adjacent to the reclamation itself. The detailed investigations should take this into account.

The effect of the reclamation on the groundwater regime should also be investigated. In addition, the effects of climate change over the life of the reclamation should be investigated and incorporated in the design.

Marine-frontage related uses, such as piers, wharves, cargo handling areas, seawater intakes, stormwater and sewerage outfalls, need to be identified at an early stage. The proposed locations of such structures, areas and facilities will need to be known, at least in general terms, prior to the detailed investigations referred to above.

Locations of roads, bridges, railways, buildings, major stormwater culverts and sewers, and their respective tolerance of settlement will influence the choice of fill to be used in different areas of the reclamation and the proposed reclamation method. Where possible, reclamation should be designed for full flexibility of land use (see Section 2.2).

A check should also be made to see if there will be any archaeological impact due to the reclamation.

2.5 Formation Level

Before deciding on the formation level for a reclamation project, the following aspects should be considered:

- Availability and cost of fill.
- Urgency of the land development.
- Existing ground, road and drain levels of adjacent developments.
- Residual settlement of the fill and underlying marine and alluvial soils.
- Type of seawall and the extent of wave overtopping expected.
- Normal and extreme water levels due to tides combined with storm surge.
- Proposed land use of the reclamation and the sea frontage.
- Possible long-term increase in mean sea level.
- Implications of climate change over the design life, including sea level rise and changes to wave conditions.

It is relatively easy in most cases to ensure that the seawall cope level and reclamation level are higher than the extreme still water level corresponding to a return period of 100 years, or even 200 years where flooding would cause substantial loss of life and damage to property. However, it is not practical to design a seawall, even with the addition of a wave wall, to effectively prevent overtopping from waves during extreme events. A certain degree of overtopping, particularly with vertical-face seawalls, is to be expected, and the drainage immediately behind the seawall must be designed to accommodate this if major flooding is to be avoided. Although wave walls can help to control or reduce extensive overtopping, careful detailing is required at discontinuities such as piers, wharves, pumphouses and landing steps to avoid significant overtopping through these areas.

In addition to determining the site formation level, due considerations should also be given to the impact on the drainage provisions of the existing adjoining lands where the formation level of the reclamation is higher than that of the adjoining lands. Appropriate drainage provisions will need to be considered to avoid flooding or overloading the existing drainage system due to the level difference.

In view of the above factors, all reclamations should be considered individually. The consequences of adopting a particular reclamation level should be fully investigated before determining the most appropriate level in each case.

2.6 Reclamation Methods

2.6.1 General

Two main reclamation methods have commonly been used in Hong Kong, namely the drained method and the dredged method. The drained method leaves the soft marine deposit in place, and the consolidation is usually accelerated by the use of vertical drains with or without surcharge preloading. The dredged method involves the removal and disposal of soft marine deposit before placing the fill. Both methods are described in this Chapter, with comments on particular aspects, advantages and disadvantages of each. Removal of sediment is generally discouraged unless there is strong justification (see WBTC 3/2000 (WB, 2000)). This must be allowed for in the reclamation programme.

In recent years, there is a tendency of not removing the sediment in reclamations, mainly on environmental reasons. Apart from the drained method, other types of soil improvement techniques can be considered. These are described in the ensuing paragraphs.

Another method used in the past is the displacement method. This method involves direct tipping of fill from trucks onto the seabed, without removal or treatment of the marine deposits. However, this method can result in mud waves and areas with marine deposits trapped under the fill. Long-term differential settlement can be a problem due to consolidation of these trapped marine deposits. This method should not be employed unless the reclaimed land can be left for decades without development.

2.6.2 Drained Method

For reclamation by the drained method, marine deposits and any underlying soft alluvial clay are left in place. The load applied by placing fill on the seabed causes an increase in the pore water pressure within the clay and silt layers. This excess pore pressure gradually dissipates, the loading is transferred to the soil skeleton and settlement takes place. The rate of dissipation of the excess pore water pressure in the soil (consolidation) depends on the layer thickness, permeability (particularly whether there are inter-bedded sandy layers), and the loading applied. A thorough site investigation, including careful ground investigation, is essential in order to ascertain the thickness of the marine deposits and underlying layers, and how they vary across the site, and the strength parameters and consolidation parameters of the various layers.

Preloading or surcharging is an effective method of increasing the rate of consolidation of thick clay and silt deposits. It is usually applied by stacking fill material, but vacuum methods have also been used elsewhere. The objective of preloading is to induce settlement greater than that predicted under the service loading. When this settlement has occurred and the surcharge is removed, rebound occurs but this follows the expansion curve (see Section 4.3 in Chapter 4). Settlement under subsequent re-loading follows the recompression curve instead of the virgin compression curve. The time at which surcharge can be removed can be determined by careful monitoring of settlements and excess pore pressures in the clay and silt deposits.

Preloading is often used in combination with vertical drains. Vertical drains are drainage conduits inserted into compressible soil of low permeability to shorten the drainage path for pore water in the soil matrix. They enable the pore water to drain horizontally to the conduits, and then vertically through the conduits to the free draining layer provided on top of the deposits, and to more permeable underlying soils, where these exist. However, vertical drains may not be necessary if the marine deposits contain inter-bedded layers of free draining granular deposits.

2.6.3 Reclamation Sequence of Drained Method

Drained reclamation is usually carried out in the following sequence (see Figure 1):

(1) Laying of geotextile on the seabed

Geotextile may be laid on the seabed to separate the fill from the underlying soft marine deposits, preventing migration of fines. It also enhances the stability of the underlying marine deposits in supporting the loading of the reclamation fill.

(2) Deposition of blanket layer

This blanket should consist of free draining granular material, placed in a series of thin layers in a controlled manner to avoid disturbing the underlying deposits until a sufficient thickness, generally 2 m, is achieved. This granular layer works with the vertical drains to enable drainage from the clayey deposits. It also acts as a capping layer to spread the load from the fill during the filling operation. If vertical drains are to be installed by marine-based plant, the material for the blanket layer and geotextile should allow easy penetration of the mandrel.

(3) Installation of vertical drains

Prefabricated paper or plastic band drains are most commonly used in local conditions because of their resistance to shearing (Premchitt & To, 1991). They can be applied economically because of their ease of fabrication, storage and installation. A triangular pattern of band drains spaced at between 1.5 and 3.0 m is normal. For a typical site with 5 to 10 m thickness of marine deposits, primary consolidation will be completed within one to two years compared with 10 to 20 years if no vertical drains are used. To commence the acceleration of consolidation earlier, the band drains are usually installed over water using special marine plant through the blanket layer of free draining granular material prior to fill placement.

The following factors should be considered when choosing the type of band drains:

- Tensile strength of the drain to withstand the stress induced during installation.

  Transverse permeability.
- Soil retention and clogging resistance.
- Vertical discharge capacity under confining pressure.
- Performance in folded condition.
- Durability.

General information on the properties and methods of testing prefabricated vertical band drains are summarized in Appendix A.

Installation of vertical drains can be carried out over water using marine plant or after reclamation using land plant. The former is usually adopted for reclamation over deep water whereas the latter is used in shallow water where marine plant access is restricted. Preboring may be necessary to by-pass obstructions for installation of vertical drains.

(4) Controlled thin-layer placement

Controlled even placement of thin layers of fill on the reclamation site is necessary to avoid shear failure of the underlying marine deposits and the formation of mud waves. Placement can be by bottom-dump barges, hydraulic filling or grabbing. An initial thickness of no more than one metre of fill is usually required with subsequent layers increased as appropriate. Layers should be staggered where possible, with control being exercised to ensure a sufficient leading edge for the underlying layer in relation to the layer being formed.

In any case, stability analyses should be carried out to check the factor of safety against shear failure or significant mud wave formation during fill placement. The following need to be specified:

- Maximum thickness of individual fill layers.
- Distance and the slope of the leading edge.
- Minimum distance from leading edge and maximum height of stockpiling of fill.

Reclamation using the drained method requires tight control on site. A detailed method statement from the contractor is required and close supervision of the filling sequence is necessary. Instrumentation for monitoring, including piezometers, inclinometers, sub-surface and surface settlement measurement systems, is essential to ensure the reclamation stability during fill placement and to check the progress of the settlement. Sampling and testing of the clay and silt deposits at various stages may be necessary to verify the assumptions made in the design, to ensure continued stability.

2.6.4 Fully Dredged Method

In this method, all marine and alluvial clays or silts are removed by dredging and replaced with fill. Normally, dredging is stopped when sandy alluvium or weathered rock has been reached, as determined by detailed ground investigation. The basic method is relatively simple but can be expensive where thick layers of soft deposits exist. Its main advantage is that settlement of the free-draining fill will take place more quickly and is more predictable, but it involves marine disposal of dredged sediments. Such disposal, particularly for contaminated mud, may be severely problematic. Chemical testing of the sediment is required to determine the level of contamination and thereby determine whether open sea disposal is acceptable or whether confined marine disposal or additional treatment is required. Tests normally comprise the analyses of heavy metal and organic compounds but biological testing will also be required if the chemical tests reveal that the sediments are contaminated. Detailed sampling and testing requirements are given in WBTC No. 3/2000 (WB, 2000).

2.6.5 Partial Dredged Method

Partial removal of marine or alluvial deposits, leaving the lower, stiffer or stronger deposits in place reduces the dredging and fill quantities compared to the fully dredged method. However, it should be noted that better control is required than for full removal for the final trimming in dredging and for initial fill placement, to avoid future differential settlement.

The remaining layer of marine or alluvial deposits will consolidate by vertical drainage upward through the fill and downward, if the layer beneath the soil is sufficiently permeable. The construction programme must still allow time for this consolidation and band drains with preloading may still be required if the available time is short.

The extent of marine or alluvial deposits to be left in place must be decided after investigation and is subject to detailed design. It will depend on the amount of settlement predicted to occur after the completion of the reclamation compared to the magnitude of differential settlement which can be tolerated in the particular situation.

2.6.6 Water Depth

Shallow water areas, with seabed level higher than about –2.5mPD, may pose restrictions to the above reclamation methods, which may include the following:

- Temporary access channel may need to be dredged in order to allow marine plant to access the reclamation site to carry out dredging works or other reclamation activities.

- Controlled thin layer placement by bottom dumping in the drained method may not be workable in shallow water because the restricted water depth will prevent the normal functioning of the bottom opening devices of the dumping barges. Placement in thin layers by grabbing will be necessary but the progress will be slower. A diagrammatic comparison of the difference of the fill placement sequence in deep water and shallow water is shown in Figure 2. Normally, filling can be executed by bottom dumping up to about –2.5 mPD; above –2.5 mPD, bottom dumping is not possible because of insufficient water depth. In deep waters, fill placement above –2.5 mPD may be carried out by end-tipping directly because the fill already placed by bottom dumping is usually thick enough to spread out the fill load. In shallow water, however, the fill layer placed by bottom dumping may not be thick enough that placing of fill above –2.5 mPD by end-tipping may induce instability. Fill placement by grab may be adopted but will result in slower progress. Both cases should be subject to stability analysis and the rate of fill placement should be controlled to avoid mud waves.

- Insufficient water depth may prohibit the access of the marine plant to install vertical drains over water. In such cases, the vertical drains may need to be installed by land plant after the reclamation is above water.

These restrictions will affect the reclamation sequence and should be taken into consideration in determining the method and programme of the reclamation.

2.6.7 Deep Cement Mixing Method

(1) Introduction of Deep Cement Mixing

The principle of deep cement mixing (DCM) is based on chemical reactions between clay and chemical agents. Ordinary Portland Cement and Portland Blast Furnace Cement, etc. are the most commonly used admixture stabilizers. The purpose of mixing chemical agents with the soil is to improve the stability, strength and stress-strain properties of the soil. The stabilization mechanism generally involves the following chemical reaction processes :

- Cement reacts with the pore water of soft clay to form a series of hydrates.
- Hydrates exchange ions with clay particles and form large conglomerates.
- Clay particles react with the excess calcium ions from the hydration process and are bonded together.

The design considerations and design approach of DCM method for the foundation of seawalls and breakwaters are given in Section 4.5.2 and Section 4.6.2 of Part 4 of the Manual respectively. For reclamation, designer could consider, apart from conventional reclamation methods (including drained method and dredged method), using DCM as one of the non-dredged methods to form land by strengthening in-situ soft marine sediment.

(2) Advantages of Deep Cement Mixing

The advantages of DCM method as compared with conventional reclamation methods are :

- DCM method solidifies the soft marine sediment by mixing it with cement slurry to form DCM columns in a shorter time frame to support the seawall and filling materials above, thus reducing the time required for reclamation. The reclaimed land could be delivered for early development.
- DCM method is an environmentally friendly method as it does not require dredging of marine sediment and importing materials for filling back the dredged area. DCM method can also significantly reduce the consolidation settlement arising from conventional drained method. Hence, the demand for importing fill materials would be reduced and disposal of dredged marine sediment would also be minimized.
- DCM is flexible in application because the amount of stabilizing agent and form of treatment can be adjusted to suit different soil properties and engineering requirements.

(2) Consideration of Deep Cement Mixing

The following considerations should be taken into account in the choice of reclamation method :

- The cost of DCM method may be higher than that of a conventional reclamation method.
- Stringent quality control and monitoring is required during the mixing process to ensure that the required strength is developed in the soil. It may be necessary to carry out field trials to obtain or verify design parameters and construction method statement for practical application, such as an optimal site - specific soil to cement ratio, water cement ratio, blade rotation number, etc.
- The rotating blades of the DCM machine may not work properly if obstructions of size larger than 250 mm are encountered during the mixing process.
- Investigations should be carried out to assess the possible environmental impacts associated with marine application of DCM and to determine if mitigation measures are necessary for a particular site. Site trial demonstration may be required. The time implication should be considered.
- It does not work well in certain soils, notably those which have a high organic content and acidic soils (Suzuki, 1982).
- It may not be applicable for very stiff or very dense ground or for treatment depth more than 40 m for land construction (Bruce, 2000) and 70 m for marine construction (Kitazume and Terashi, 2013).
- The change of unit weight of deep mixed zone is not significant. Therefore, no noticeable additional surcharge will be induced on the underlying soil strata due to the DCM stratum.
- The design of DCM works shall take into consideration the future land use and control the residual settlement to the acceptable level.
- The design consideration includes the geological conditions, treatment depth, design strength of DCM columns, spacing of DCM columns, filling depth and overburden pressure etc.
- The layout of DCM works shall be designed to cater for any horizontal stress arisen during construction and/or in the permanent state.
- The future developers of the land should be alerted to the modified ground conditions as the variability of sub-soil properties shall be taken into account during their foundation design and construction works.
- For pile foundation design, designer shall take into consideration the properties of DCM - treated soil in determining piling design layout and construction method, with a view to penetrating through DCM-treated soil to a deeper stratum for assuring pile capacity and reducing settlement of the superstructure.
- If DCM stops penetration in marine sediment without fixing on hard stratum (i.e. floating type improvement), some amounts of ground settlement will take place in the marine sediment beneath the DCM.
- The operation of DCM may cause heave and horizontal displacement of ground. The amount of ground deformation depends on the improvement area ratio and construction sequence.
- Spoils from DCM operation require treatment and/or disposal.

2.6.8 Other Soil Improvement Techniques

Soft deposits may also be strengthened in-situ by vibro-replacement (stone columns) or by soil mixing techniques including lime columns or any other soil improvement techniques. Other form of seawall construction methods or combination of methods may also be considered (e.g. using cofferdam steel cells sinking to hard stratum). Designers should consider the cost implications and possible effects on piling and excavation in future land development. These specialist techniques may require detailed assessment to demonstrate their viability.

2.7 Fill

2.7.1 General

Fill for reclamation includes public fill, marine sand fill and crushed rock, but public fill and marine sand fill are the most commonly used types of fill in local conditions. For most reclamation works, the choice of fill is largely dependent on availability and cost. In order to maximize the benefit of available fill sources, flexibility for combining different types of fill should be allowed in a project. The Public Fill Committee and Marine Fill Committee, under the chairmanship of the Director of Civil Engineering, should be consulted for the use of fill and related procedural requirements during the planning of reclamation projects.

2.7.2 Public Fill

Because of the shortage of areas to accommodate the public fill generated by the construction industry, priority should be given to its use. It is also the government policy to maximize the use of public fill in reclamation projects. The main advantage of using public fill is the benefit to the community. Reclamation using public fill provides sites for the disposal of the inert portion of construction and demolition material from private and public developments and demolition sites. This ensures that good use is made of such material rather than it becoming a wasted resource.

Particular aspects to be noted in the design of the reclamation when using public fill are:

- The rate of supply of public fill is not easily controllable, being dependent mainly on programming of engineering works and development projects. It is quite usual for the number of trucks per day to fluctuate, and the delivery rates can vary widely from about 1,000 to 15,000 m^3 per day. The impact of the fluctuation of the rate of supply on the reclamation programme should be carefully assessed.

- Strict site control is necessary to ensure that public fill does not contain unsuitable material such as marine mud, household refuse, plastic, metal, industrial and chemical wastes, animal and vegetable matter and other material considered unsuitable for reclamation.

- Rock and concrete over 250 mm would impede subsequent piling, and has to be broken down to this size or placed in areas zoned for open space and roads.

- The use of public fill will generate a large number of delivery trucks at and around the entrances and exits of the reclamation site or barging points. The route of the trucks going to and leaving the site or barging points should be properly considered to avoid unacceptable disturbance to the existing traffic. Sufficient space should be allowed on site to cater for queuing of trucks and adequate traffic signs should be provided to give clear instructions to the truck drivers. Transport Department, Police, District Office, District Councils and other relevant parties should be consulted in planning the traffic arrangement.

- Where direct land access to the site is not available, or where controlled thin layer fill placement is required in a reclamation using the drained method, contract arrangements should ensure that barges are available at all times to avoid the need for temporary stockpiling of public fill at the barge loading site. At the reclamation site, public fill will either be bottom-dumped by barges or placed by grab, depending on the available water depth. Usually, completion of the reclamation to the required formation level is not possible using marine plant and direct grab placement, and the use of temporary stockpiles and final placement of fill using site trucks and spreading plant is common.

- Public fill can be directly end-tipped onto the reclamation under the control of site supervisory staff when the dredged method is adopted or when the fill thickness or profiles already deposited on site in the drained method is adequately safe against slip failure or mud wave formation. In either case, verification of stability by calculation is required. Normally, end tipping can be carried out in two stages, firstly to a general level just above the high water mark and later to the formation level of the reclamation.

- Sampans should be employed to pick up floating material from the water within the reclamation site. Such material is unsightly and can be hazardous to shipping if allowed to drift into open waters. Floating refuse containment booms should be used to surround the site to contain the debris and to facilitate removal by the sampans.

2.7.3 Marine Fill

The use of marine sand fill can be economically viable for a reclamation project where a marine borrow area is within a reasonable distance of the site and where the size of the project justifies the use of sophisticated dredgers, which have high mobilization costs. Plant such as trailing suction hopper dredgers may dredge marine sand fill at relatively low costs, depending on the thickness of the overburden at the marine borrow area. The rate of formation of reclamation can be very rapid compared to the use of other types of fill, and often several dredgers can be used on the same site, if necessary. These dredgers can deposit marine sand in the reclamation by bottom dumping or by pumping, depending on the access and water depth available.

The maximum fines content (smaller than 63 ∝m) of the marine fill used in reclamation should always be less than 30% but for hydraulically placed fill, it is usually limited to about 10%. The specification should suit the reclamation design. If the marine sand is to be densified by vibrocompaction, the maximum fines content requirement will need to be more stringent (see Figure 3). This requirement applies to fill as delivered to the reclamation site, and not necessarily to in-situ material in the borrow area. Sampling on-board dredgers to check the grading of material is one method to confirm that the required specification for fill delivered to the site is satisfied. Samples can also be taken at the reclamation site, to check if segregation of material has occurred during placing, with a view to ensuring that the fines are well distributed within the reclamation. For proper control of the works on site, the tests for the suitability of marine fill and the uniformity of the as-placed material should be stated in the specification.

2.7.4 Crushed Rock

Crushed rock from local land sources should not normally be used for reclamation in view of the shortage of public filling capacity, but should be used as foundation materials or processed to produce aggregate products, as far as possible. If available for reclamation, it is usually a by-product of works projects involving large quantities of soil or rock excavation and removal, having a project programme that ties in with that of the reclamation project. Where crushed rock over 250 mm is used, it should be placed in areas where no building development will take place, to avoid impeding piling or excavation works in the future.

2.8 Fill Treatment

2.8.1 General

Measures to improve the settlement characteristics of reclamation fill include densification by static methods (such as surcharge preloading and vacuum preloading) or dynamic methods (such as vibratory probe techniques and dynamic compaction). The choice of a particular method is dictated by the degree of improvement required, the depth of fill to be treated, the proximity to existing structures or facilities, and the relative cost benefits. These fill treatment measures may be included in the reclamation programme, after considering:

- Whether the future development plan of the reclaimed land is known to the designers in the design stage and whether the type, dimension and time of construction of any structures to be built on the reclaimed land are known.
- Cost-benefit and programming considerations.
- Effect of the densified fill on future piling, excavation and other development
- activities.

2.8.2 Surcharge Preloading

Surcharge preloading can be used to accelerate settlement of fill that would otherwise occur more slowly. Monitoring the fill densification achieved by surcharge preloading is essential. This can be done by field measurement tests (see Section 2.8.5) and by testing samples obtained at intervals by drilling. The surcharge should only be removed when the required settlement or increase in strength has been achieved.

2.8.3 Dynamic Compaction

Dynamic compaction involves repeated dropping of heavy weights onto the ground surface. Large amounts of energy are transferred to the soil in the form of impact force and waves, particularly shear waves. This results in a densely packed particle arrangement. Dynamic compaction is suitable for use in most soils except cohesive soil below the water table.

The pounders used for dynamic compaction may be concrete blocks, steel plates, or thick steel shells filled with concrete or sand, and may range from one or two up to 200 tons in weight. Drop heights up to 40 m have been used. Dynamic compaction can also be carried out underwater.

The major parameters to be determined in planning a scheme of dynamic compaction are the extent and depth of treatment possible and the degree of improvement required. The former is related to the locations and structural characteristics of settlement-sensitive development on the reclamation as well as the possibility of damage to nearby buildings or facilities due to the compaction works. The latter normally refers to the required relative density measured by direct and indirect tests and is related to the required bearing capacity or tolerable settlement under working load of the proposed structures. A site trial can be performed to determine the optimal values of these parameters. Verification may be achieved by carrying out compaction compliance tests (Section 2.8.5).

2.8.4 Vibrocompaction

Vibrocompaction is used for granular soils, in particular sand. It is carried out by penetration and controlled retraction of a vibrating vibroflot in the soil. Under a state of high frequency vibration and with the assistance of a water jet or compressed air, the intergranular forces between the soil particles are temporarily nullified and liquefaction occurs, allowing the particles to be rearranged in a denser matrix. The degree of compaction is controlled by the energy input and spacing of the compaction points.

A centre to centre spacing of 2.5 to 4 m is typical for clean marine sand fill. Compaction depth of 25 m is common but up to 35 m has been achieved. A compaction trial can be performed to determine the optimal values of spacing and depth.

Unlike dynamic compaction, which can be used on most types of fill, vibrocompaction is only applicable to granular materials of certain grading properties. A graph showing the suitability of materials for vibrocompaction with respect to the grading characteristics is shown in Figure 3.

Vibrocompaction cannot effectively compact the surface few metres of fill and therefore separate compaction of the surface layer will be required.

Considerations when planning the vibrocompaction are similar to those of dynamic compaction. These include determination of the treatment extent and depth as well as the degree of required improvement. The specification for the performance of fill densification by vibrocompaction is also similar to that of dynamic compaction, given in Section 2.8.5.

2.8.5 Specification for Fill Densification

The degree of improvement or performance of fill densification may be revealed by the relative density achieved. Relative density is a useful way to characterize the density of granular soil and is defined as:

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100\%$$

or
$$D_r = \frac{\gamma_{d\max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \times 100\%$$

where $e_{\min}$ = void ratio of the soil in densest condition

$e_{\max}$ = void ratio of the soil in loosest condition

e = in-place void ratio

$\gamma_{d\max}$ = dry unit weight of soil in densest condition

$\gamma_{d\min}$ = dry unit weight of soil in loosest condition

γ_d = in-place dry unit weight

Relative density can be correlated with the results of in-situ tests such as Standard Penetation Tests (SPT) and Cone Penetration Tests (CPT). Typical correlations between CPT cone resistance and SPT value with relative density and friction angle of sand, are shown below:

	Very Loose	*Loose*	*Medium*	*Dense*	*Very Dense*
SPT N-value (blows/0.3m)	0-5	5-10	10-30	30-50	>50
CPT cone resistance (MPa)	<2	2-4	4-12	12-20	>20
Relative density (%)	<20	20-40	40-60	60-80	>80
Friction Angle (degree)	<28	28-30	30-36	36-41	>41

The CPT is usually preferred because it is a relatively more precise test compared to the SPT and can provide data over the depth of the fill at a much faster rate than the SPT. As a general indication, an average CPT greater than about 8 MPa should be achieved for areas designated for roads, drains and box culverts where fill treatment has to be carried out. The depth/density profile requires careful consideration to avoid over or under specification.

2.8.6 Other Fill Treatment Methods

The vibro-replacement and soil mixing techniques mentioned in Section 2.6.7 may also be applied to fill treatment. These techniques are useful where culverts, pipelines or low-rise structures need to be supported.

2.9 Seawall Construction

Guidance on the design of seawalls, normally accompanying a reclamation project, is given in Part 4 of the Manual – Guide to Design of Seawalls and Breakwaters.

The reclamation sequence can be affected by the construction of the seawalls. Some of these aspects which should be investigated in the reclamation design include:

- In the case of a drained reclamation, filling works adjacent to an uncompleted seawall foundation can adversely affect the stability of the reclamation and lead to slip failure. The foundation should be completed before fill placement adjacent to it proceeds.

- Rapid fill placement behind a recently completed seawall, particularly when clay or silt remains at the bottom of the seawall foundation, can induce instability in the reclamation, as the dissipation of the excess pore water pressure due to the fill loading cannot normally be completed within a short period.

- Consideration may be given to forming a bund after the construction of the seawall and placement of filter material, using selected granular material where possible, along the line of and immediately behind the seawall. Such a bund assists in stabilizing the seawall and its foundation if mud waves occur during filling.

2.10 Environmental Impacts

All reclamation works will induce impact on the environment, the extent of which should be assessed carefully to determine if mitigation measures are required to minimize the impact. The potential impacts of a typical reclamation project are:

- Dredging – noise, sediment plumes, release of nutrients or contaminants from dredged sediments, dissolved oxygen depletion, habitat destruction and ecological impacts.

- Marine borrow works – the same as due to dredging works, and the possibility of reduction or removal of the natural supply of sand to existing beaches.

- Land borrow works – noise, dust and smoke generation, and visual impact.

- Mud disposal – release of nutrients or contaminants from deposited sediments; other impacts similar to those due to dredging works.

- Fill delivery – noise and dust generation.

- Fill placement – noise, dust and smoke generation, and water quality impact.

- Final landform – interference in tidal flow, wave and sediment transport patterns, siltation, scour and reduced dispersion or dilution of discharges, water quality and ecological impact, elevation of groundwater levels uphill, and possibility of causing erosion of the shoreline of existing beaches.

Guidance on environmental impact assessment during the planning stage of the reclamation project, including the approach and assessment methodologies, is given in the Technical Memorandum on Environmental Impact Assessment Process (EPD, 1997). The recommendations of the environmental impact assessment study, such as the implementation of environmental mitigation measures and monitoring and audit programme, may affect the overall development of the project. Therefore, proper allowance for these factors should be made when preparing the reclamation programme.

2.11 Drainage

A reclamation project may cause adverse impacts on drainage and flooding by altering the existing drainage paths and regime, or by increasing the burden on the existing drainage system. Existing drainage outfalls will need to be extended and temporarily diverted away from the line of the extension while reclamation is carried out. A drainage impact assessment is also necessary at an early stage of the planning and design in order to assess potential drainage and flooding problems. It must determine the necessary temporary drainage diversion and permanent drainage measures to ensure acceptable drainage performance in areas upstream, adjacent to and inside the reclamation during and after the construction of the reclamation. Close liaison with Drainage Services Department is necessary for the drainage impact assessment. For details of drainage impact assessment study, WBTC No. 18/95 (WB, 1995) should be referred to.

During reclamation, care should be taken to check for existing outfalls, intakes and drains along the lines of the existing shoreline, particularly at low tide. Any outfalls, intakes or drains which do not appear on the drawings should be recorded immediately and investigated with the appropriate authority. Temporary channels should be provided as necessary across the reclamation as a short term measure to avoid any delay. This aspect should be indicated in the contract.

3. STABILITY

3.1 General

The reclamation sequence, dredged level, spacing of vertical drains, magnitude and duration of surcharging and so forth are largely determined by stability and settlement criteria. This chapter focuses on the stability of a reclamation and methods of stability analysis. Settlement is discussed in Chapter 4 of this part of the Manual.

3.1.1 Factor of Safety

If the factor of safety against slip failure is kept above a minimum level of unity at all stages of filling, instability should not occur. However, the choice of the factor of safety to be used is a matter of professional judgement as to the acceptable consequences of failure, in terms of risk to life and economic consequences. It is wise to allow a safety margin for unknown factors since ground conditions are variable and ground investigation samples only a very small portion of the ground. Each reclamation must be assessed on its merits and, where justified, a higher factor of safety should be used. Reference should be made to WBTC No. 13/99 (WB, 1999) on the choice of factor of safety for temporary works.

3.1.2 Strength of Materials

(1) Marine and Alluvial Deposits

Marine and alluvial deposits are variable in grading and a thorough ground investigation is necessary in order to determine whether layering is present and the grading of the layers. If the deposits are predominantly soft clay, their behaviour may be predicted by assuming that they are undrained. This is justified because the permeability is low, and the time for construction is usually short compared to the time for full drainage. Stability may be analyzed by a total stress method using an appropriate profile of undrained strength c_u.

The field vane shear test is commonly used for the in-situ determination of the undrained strength c_u of soft clays, with the application of correction factor ($\propto$). Thus,

$$c_u = \mu (c_u)_{FV} \tag{3.1}$$

where $(c_u)_{FV}$ is the measured undrained strength from field vane.

According to Bjerrum (1972), $\propto$ is a function of the plasticity index I_p (see Figure 4).

The plasticity index I_p can be determined from laboratory tests. The profile of shear strength for each layer of clayey deposit can then be determined from the above relationships.

The undrained shear strength, c_u, can also be determined from laboratory triaxial tests performed under undrained conditions. The testing procedure can be found in Geospec 3 (GEO, 2017). Shear strengths obtained from laboratory triaxial tests are usually lower than those from field vane shear tests, due to sampling disturbance. The disturbance can be reduced by using piston sampling but the vane shear test results are generally considered to be more reliable, particularly for sensitive clays (GCO, 1987). Cone penetration tests can also provide a reliable, quick method of investigating the soil structure, as continuous profiles are obtained. The results of the cone penetration tests can be correlated with the undrained shear strength; methods of correlation are given in Lunne (1997).

(2) Strength of Reclamation Fill

Fill is usually placed in a loose state in a reclamation although it may become densified later under the weight of the overlying fill layers (including surcharge, if used). The strength of reclamation fill adopted in the stability analysis should therefore reflect the strength appropriate to the state of compaction of the fill at the time.

Typical unit weights and friction angles for sand fill and decomposed granite or similar types of fill for preliminary stability analysis during fill placement are summarised in the following:

Bulk density:	19 kN/m^3
Friction Angle:	30°
Cohesion:	0

3.1.3 Stability Analysis

For detailed analysis of the stability of each stage of the reclamation, the methods recommended in Section 5.3.5 of the Geotechnical Manual for Slopes (GCO, 1984) should be used.

The geometry of reclamation, including side slope, layer thickness and dimensions of berms (if any) should be so designed that failure does not occur. The practical verification of this requires checking of the equilibrium of the soil mass along a sufficient number of arbitrarily selected failure surfaces to find the minimum value of the factor of safety for each reclamation stage.

The principle of method is illustrated in Figure 5. The factor of safety associated with the selected failure surface is defined by:

$$F = \frac{\sum R\tau_f l}{\sum Wx} \tag{3.2}$$

where	R	=	Moment arm for τ_f.
	τ_f	=	Shear strength along the selected surface.
	l	=	Length of vertical slices along the selected surface.
	W	=	Weight of vertical slices.
	x	=	Moment arm for W.

The slip surface in Figure 5 is for illustration only. Designers should consider all possible slip surfaces, either circular or non-circular ones, in the analysis. Stability analysis is usually performed using a computer program. Several are available in the market.

3.2 Drained Reclamation

3.2.1 Stability during Construction

The stability of a reclamation during filling is particularly important where the underlying soft marine and alluvial deposits are left in place. Improper placement of fill on such soil could lead to failure and have significant programming, financial and life consequences. Therefore, any proposed reclamation methodology and sequence, as well as subsequent changes, must be subject to stability analysis before it is executed. Fill placement in shallow water onto thick marine mud is likely to cause mud wave formation unless it is carried out carefully, and stability at each interim construction stage should be checked during the design phase.

(1) Simplified Analysis Method for Short-term Stability

The simplified stability analysis method described in this section may be used to provide a quick preliminary assessment of the stability of the leading edge of the reclamation.

Jewell (1987) and Yeo & Woodrow (1992) considered that an embankment on soft clay is essentially a problem of bearing capacity. Due to the loading from the fill, the clay layer at the bottom of the embankment experiences both vertical stresses due to surcharge loading and horizontal stresses. Therefore, the loading from an embankment onto the soft soil creates not just vertical stress but also an outward shear stress. The combined effect of vertical stress and the outward shear stress is to reduce the resistance of the foundation to the applied vertical stresses. If the foundation cannot support the lateral shear stresses, it will deform laterally. The lateral displacement may lead to cracking of the fill, destroying whatever contribution the fill strength makes to the overall stability of the embankment.

For an embankment constructed on soft clay of uniform strength and limited depth with respect to the embankment width (see Figure 6), Jewell (1987) derived the following expression:

$$\frac{F\gamma H}{c_u} = 4 + \frac{(1+\alpha)L_e}{D} \tag{3.3}$$

where

F	=	Factor of safety.
H	=	Height of embankment fill.
γ	=	Unit weight of embankment fill; for fill underwater, the submerged unit weight should be used.
L_e	=	Length from the toe to the crest of embankment.
c_u	=	Undrained strength of soft clay.
D	=	Thickness of soft clay.
α	=	Ratio of the shear stress developed at the base of the embankment to the available c_u.

For an unreinforced embankment:

$$\alpha = -\frac{K_a \gamma H^2 F}{2 c_u L_e}$$

where K_a = Coefficient of active pressure of fill.

(2) Detailed Stability Analysis

The detailed stability analysis method for checking the stability at each interim construction stage is covered in Section 3.1.3.

(3) Use of Geotextile

Where used, geotextile is normally laid on the seabed before reclamation fill is placed. It provides separation between the fill and the underlying marine mud preventing the punching of the fill into the seabed and helps to prevent the creation of mud waves. However, the benefits of the geotextile may be negated by improper overlapping or inadequate seaming at the joints. Close site supervision and careful inspection after laying is required to ensure the proper functioning of the geotextile.

(4) Monitoring

Monitoring by means of instrumentation plays an important role in detecting potential failure during filling. The instrumentation data collected on site can be used to estimate the settlement, lateral deformation and pore water pressure change of the underlying marine mud, alluvial clays and silts. Details are covered in Chapter 5 of this part of the Manual.

(5) Miscellaneous Considerations

The following should also be considered:

- Site history – Pre-construction site investigation should be well planned and executed to provide up-to-date and sufficient information for the proper design of the reclamation. During the site investigation, if there are unusual irregularities in the seabed topography or subsoil layers, the designer must determine whether these are signs of distress caused by past works. If there is evidence that the marine mud in the site has been disturbed before, tests to determine the remoulded strength must be carried out. The remoulded shear strength is more appropriate for stability analysis in this situation. The disturbed mud profile should also be used in the analysis.

- Reclamation sequence – Fill should not be placed until seabed preparation works have been completed, to the extent that fill deposition will not adversely affect the stability. Preparation works include laying of geotextile where

- appropriate, deposition of the sand blanket and installation of band drains. For areas in the vicinity of proposed seawalls, placement of fill should not commence until completion of the seawall foundation.

- Stockpiling of fill – The reclamation sequence should be carefully planned to make sure that the programmes of seabed preparation and seawall construction are compatible with the intake rate of fill. Stockpiling must not be allowed where it might cause failure, for instance, close to recently constructed seawalls or the edge of the reclamation.

- Site supervision – Stringent control should be exercised on the placement of fill in thin layers, particularly on the initial layers. Fill placement in shallow water onto thick marine mud is very vulnerable to mud wave formation. Stability at each interim construction stage should be checked.

3.2.2 Long-term Stability of Drained Reclamation

Since reclamation is ultimately enclosed by seawalls, the long-term stability of a reclamation depends on whether the seawalls are capable of retaining the fill. Guidance on design related to the stability of seawalls is given in Part 4 of the Manual – Guide to Design of Seawalls and Breakwaters.

3.3 Dredged Reclamation

The principles and methods for analyzing the stability of a dredged reclamation are in general the same as those of the drained reclamation. Since the soft, compressible sub-soils such as marine mud or clayey alluvium deposits are removed, the stability during construction is normally less critical than that of a drained reclamation. However, if there are existing structures or developments adjacent to the reclamation site, the stability of the temporary slopes of the dredged surface should be checked to ensure that the factor of safety is adequate (see Section 3.2.1).

For the long-term stability of seawalls enclosing a dredged reclamation, Part 4 of the Manual – Guide to Design of Seawalls and Breakwaters should be referred to.

3.4 Remedial Works

If, despite all the precautions described previously, shear failure or significant mud waves occur during fill placement, remedial works will be necessary. This section describes ways to rectify the damage.

Site investigation must be carried out first to identify the extent of the failure before deciding the remedial works to be undertaken. The extent of failure may be assessed by a combination of the following methods:

- Estimation of the scale and extent of mud waves from site observations and sounding surveys of seabed profile.
- Back analysis of reclamation stability based on the loading conditions before failure and soil parameters from previous site investigations to estimate the probable location and depth of failure zone.
- Carrying out post-failure ground investigation and soil testing, and comparing soil samples, profile and strength parameters obtained before and after failure.

Since any disturbed material may have a lower remoulded strength, tests such as field vane shear tests are required to provide appropriate strength parameters for the design of remedial works. In addition, disturbed mud is generally slower to consolidate than undisturbed mud and therefore the effect of disturbance on the coefficient of consolidation should be investigated before starting the design.

(1) Complete Removal

Where mud waves are generated, complete removal of all disturbed mud and replacement with fill is the quickest way of rectifying the situation. The method is often employed to remove small-scale mud waves generated in reclamation projects. The advantage of the method is the relatively small settlement in the area of the replacement fill. The disadvantages are the relatively high cost of dredging the mud, the need for a dumping ground for the dredged mud and the need for additional fill. Subsequent differential settlement may occur between the non-disturbed area and the rectified area where mud disturbed at greater depth is also removed and replaced by fill. This should be checked in the remedial proposal.

Stability should be checked to determine the factor of safety against slip failure in the adjacent reclamation if the disturbed marine mud is removed. The settlement and lateral deformation of the reclamation should be monitored to ensure safety during mud removal.

(2) Accelerated Consolidation

This method involves placing granular fill in thin layers on top of the mud wave and accelerating the consolidation of the mud with the installation of vertical drains and surcharging. This method avoids dredging of the disturbed mud and thus reduces the need to replace the void with fill. The magnitude and rate of settlement should be estimated. A stability check will be required if surcharge is used to accelerate the rate of consolidation.

As long as sufficient time for consolidation can be allowed in the development programme for the newly reclaimed land, this is an economical method of rectification, particularly where the mud wave involves movement at a deeper level.

(3) Partial Removal

Sometimes, it may be advantageous to partially remove the disturbed mud in order to accelerate the time of consolidation and to reduce the magnitude of settlement. This method is a combination of the methods mentioned above. The weak upper layers of disturbed mud are removed while lower stiffer mud is left in place. Accelerated consolidation of the remaining disturbed mud is still required. Stability and settlement calculations (see Chapter 4 of this part of the Manual) are required to check the effectiveness of the proposed remedial works.

4. SETTLEMENT ASSESSMENT

4.1 General

This chapter discusses the various components of settlement and gives general guidance on the principles of calculating the rate and magnitude of settlement. The results of calculation provide information on the effect of settlement on future land use.

It should be noted that the settlement estimated in design may differ from the actual value because of the variability of soil properties likely to be encountered in local marine conditions. The settlement should be verified with field monitoring data. Numerical computation software may assist in settlement prediction, particularly if the soil strata are complicated.

4.2 Components of Reclamation Settlement

Settlement of a reclamation will generally continue for a considerable time after completion of filling, but at a decreasing rate. Residual settlement is the amount of future settlement of the in-situ soils and fill expected to occur, from a given time. The given time may refer to the date of completion of the filling works, the date of handing over the reclamation site to another party, the scheduled development date of the reclaimed land or any other time. Hence, when describing the amount of residual settlement, it is necessary to define the start time.

For the proper assessment of settlement performance it is necessary to understand the various components contributing to the residual settlement. These settlement components are:

(1) Primary consolidation settlement

Consolidation is the gradual reduction in the volume of a fully-saturated soil due to the drainage of pore water. The speed of this process depends on the permeability. In high permeability soils this process is almost instant but in low permeability soils it continues for a long time, until the excess pore water pressure has completely dissipated. The initial excess pore water pressure is equal to the increase in vertical stress applied.

(2) Secondary consolidation

Secondary consolidation (creep) is the long-term settlement under constant effective stress. In clay, it is due to the squeezing out of the water adsorbed on the clay particles. Adsorbed water is chemically bound to the surfaces of the clay particles, so it is not free to flow under gravity. As the adsorbed water is squeezed out, the clay particles move closer and rearrange to a new equilibrium position, resulting in further deformation or compression. There are conflicting views as to whether secondary consolidation starts as soon as the loading is applied or after completion of primary consolidation.

(3) Creep of fill

Creep is also said to occur in granular fill. Some fill exhibits time-dependent compression with the rate similar to that of secondary consolidation in clays. The mechanism of such creep behaviour of granular fill is explained by the crushing of the contact points between the particles, which results in the reorientation of the soil particles and additional settlement.

4.3 Assessment of Sub-soil Settlement

4.3.1 General

This section discusses the principles of settlement analysis, based on the classical theory of Terzaghi, which describes primary consolidation resulting from a change in loading (dissipation of pore water pressures) and secondary consolidation that takes place subsequently with no change in pore water pressure. The classical theory assumes one-dimensional strain and negligible lateral deformation. Since reclamation usually covers a large horizontal area relative to the thickness of the sub-soil, these conditions are met. However, the classical theory was derived from observations of thin samples during tests in the laboratory. In reality, it has been observed in the field that thick soil layers behave differently to this classical concept. There is no clear separation of the two types of settlement and the overall result is one of gradually reducing pore pressures and reducing rates of settlement, and excess pore water pressures may still be observed near the end of primary consolidation. Hence, computations based on two components of strain, one related to changes in effective stress and the other time dependent, not operating sequentially but taking place simultaneously, may be a better model for more accurate prediction of settlement in thick soil layers. Estimation of pore water pressures will require advanced calculation methods to cater for the combination of the two components of strain.

4.3.2 Settlement due to Primary Consolidation

The ultimate primary consolidation settlement of a soil layer due to an applied loading depends on the relative magnitudes of the initial effective vertical stress acting on the soil and the effective preconsolidation pressure, and can be estimated as follows:

For $\sigma_{v0}' = \sigma_p' < \sigma_{v0}' + \Delta\sigma_v$ $$S_p = H(CR \log \frac{\sigma_{v0}' + \Delta\sigma_v}{\sigma_{v0}'}) \quad (4.1a)$$

For $\sigma_{v0}' < \sigma_p' < \sigma_{v0}' + \Delta\sigma_v$ $$S_p = H(CR \log \frac{\sigma_{v0}' + \Delta\sigma_v}{\sigma_p'} + RR \log \frac{\sigma_p'}{\sigma_{v0}'}) \quad (4.1b)$$

For $\sigma_{v0}' < \sigma_{v0}' + \Delta\sigma_v < \sigma_p'$ $$S_p = H(RR \log \frac{\sigma_{v0}' + \Delta\sigma_v}{\sigma_{v0}'}) \quad (4.1c)$$

where σ_{v0}' = Initial effective stress in the soil layer.

σ_p' = Effective preconsolidation pressure, which is the maximum effective vertical stress that has acted on the soil layer in the past and can be determined from laboratory oedometer tests.

$\Delta\sigma_v$ = Applied loading due to the fill and future imposed load on the reclamation.

S_p = Ultimate primary consolidation settlement of the layer concerned.

H = Thickness of the soil layer.

CR = Compression ratio, equal to the slope of the virgin compression portion of the ε-logσ' plot as shown in Figure 7.

= $\dfrac{C_c}{1+e_0}$

RR = Recompression ratio, equal to the average slope of the recompression portion of the ε-logσ' plot as shown in Figure 7.

= $\dfrac{C_r}{1+e_0}$

C_c = Compression index which can be estimated from laboratory oedometer tests.

C_r = Recompression index which can be estimated from laboratory oedometer tests.

e_0 = Initial void ratio of the layer.

The primary consolidation settlement of a homogeneous soil stratum at a given time *t* is given by the following equation:

$$S_{p,t} = \sum_{i=1}^{n} \Delta H \left(CR \log \frac{\sigma_{v0}' + \Delta\sigma_{v,t}}{\sigma_{v0}'} \right) \tag{4.2a}$$

If the average degree of consolidation exceeds 90%, $S_{p,t}$ may be approximated by:

$$S_{p,t} = U_t \cdot S_P \tag{4.2b}$$

where	$S_{p,t}$	=	Primary consolidation settlement achieved at time t.
	U_t	=	Average degree of consolidation at time t for a homogeneous soil stratum (see Appendix B for more details).
	S_p	=	Ultimate primary consolidation settlement.
	t	=	Time measured from the point at which fill placement commences.
	n	=	Number of sub-divided layers of the soil stratum.
	ΔH	=	Thickness of sub-divided layers (equal H/n).
	$\Delta\sigma_{v,t}$	=	Change in effective vertical stress at time t.

If vertical drains are used, the length of the drainage path is shortened and the equations must be modified (see section 4.3.4).

If preloading is adopted, the applied loading in Equation 4.1 should include that due to the surcharge load, taking into account its duration of application (see Section 4.3.5).

If the soil properties are variable over the depth, the calculation should be broken down into a number of layers, each of which represents the properties of the soil at the respective depth.

Residual settlement due to primary consolidation

From Equations 4.1 and 4.2, the residual settlement of a soil layer due to primary consolidation at time *t* is given by:

$$S_{p,t(residual)} = S_p - S_{p,t} \tag{4.3}$$

where $S_{p,t(residual)}$ = Residual settlement of the soil layer due to primary consolidation at time t.

S_p = Ultimate primary consolidation settlement of the soil layer (see Equation 4.1).

$S_{p,t}$ = Primary consolidation settlement of the soil layer achieved at time t (see Equation 4.2).

4.3.3 Correction for Construction Period

In practice, the loading due to fill is not applied to the soil instantaneously but over a period of time. The calculation of the consolidation settlement achieved within a give time therefore needs to be corrected for the loading variation during the construction period (t_c). There are several ways of dealing with this but the simplest is the equivalent load method. It may be assumed that the load due to the fill increases uniformly throughout the construction period. Thus the degree of consolidation at time t_c, the end of the construction period, is the same as if the total load due to the fill had acted constantly for a period of $t_c/2$. Hence, at any time after the end of construction, the corrected time for the calculation of the consolidation achieved is equal to the time from the start of fill placement less half the construction period ($t_c/2$).

Similarly, at any time during the construction period, the consolidation achieved may be taken as being equal to that occurring for instantaneous loading at half that time. Since filling is still in progress, the load acting is not the total load of the fill. Hence, the value of the consolidation achieved should be reduced in proportion of current load to the total load.

Thus, the primary consolidation settlement at any time t after commencement of fill placement can be determined as follows:

(1) At any time t during the construction period

Settlement = (settlement for instantaneous loading, computed using $0.5t$)
× (fraction of final fill load in place)

(2) At any time t after the construction period of duration t_c

Settlement = settlement for instantaneous loading (final fill load), computed using $t - 0.5t_c$

If the filling progress deviates significantly from the assumption of linear loading increase, the correction applied to the construction period should be adjusted accordingly. For more information on consolidation due to time-dependent loading using Terzaghi's one-dimensional consolidation theory, reference may be made to Schiffman (1958).

4.3.4 Primary Consolidation with Vertical Drains

The assessment of the rate of consolidation with vertical drains is given in Appendix B and is based on the principle that the presence of vertical drains shortens the drainage path within the soil layer, resulting in faster dissipation of excess pore water pressure (see Figure 8). Consolidation then takes place by horizontal radial drainage, thereby increasing the magnitude of the degree of consolidation given in Equation 4.2 and in turn the average degree of consolidation achieved within a given time. The assessment may need to consider the following factors, which tend to reduce the effectiveness of the vertical drains:

- Smear Effect – Installation of vertical drains may cause disturbance in the surrounding soil, decreasing the permeability and affecting the performance of the vertical drains. This is called the "smear effect", and the disturbed zone around the vertical drains is called the smear zone. The extent of the smear zone is dependent on the method of installation. For band drains, which are usually installed by a mandrel, the extent of the smear zone depends on the size of the mandrel.

- Well Resistance – Vertical drains have finite permeability with respect to the surrounding soil, which is known as the "well resistance". Well resistance depends on the theoretical amount of water the consolidating soil can expel, the actual discharge capacity and maximum drainage length of the drain.

The final magnitude of consolidation settlement is theoretically the same as that without vertical drains. Only the rate of settlement is affected. Secondary consolidation cannot be accelerated by vertical drains.

4.3.5 Primary Consolidation with Surcharge Preloading

Vertical drains and surcharge preloading are often used together to speed up the consolidation settlement. The surcharge serves to precompress the underlying soils to wholly or partially eliminate the consolidation settlement that would occur under the anticipated future loading.

Design of surcharge preloading generally involves consolidation analysis with loading variations during the project period. Such loading variations usually consist of fill placement to the formation level, construction of the surcharge mound, and removal of the surcharge mound back to formation level. The residual settlement after preloading (i.e. after removal of the surcharge mound) at time *t* may be estimated from the following formula:

$$S_{p,t(residual)} = S_p - S_{p,t} \tag{4.4}$$

where $S_{p,t(residual)}$ = Residual settlement of the sub-soil layer due to primary consolidation at time *t*.

S_p = Ultimate primary consolidation settlement of the sub-soil layer.

$S_{p,t}$ = Primary consolidation settlement of the sub-soil layer with surcharge preloading at time *t*.

Since consolidation due to every load increment proceeds independently of the consolidation due to the preceding and succeeding load increments, $S_{p,t}$ can be found by the principle of superposition.

4.3.6 Settlement due to Secondary Consolidation

Secondary consolidation of a soil layer at a given time *t* can be calculated using the following formula (Carter, 1996):

$$S_s = C_{\alpha\varepsilon} H \log\left(\frac{t_2 - t_0}{t_1 - t_0}\right) \tag{4.5}$$

where S_s = Settlement due to secondary consolidation that takes place between time t_1 and t_2.

H = Layer thickness.

t_0 = Start of the time for secondary-consolidation calculations.

t_1 = Time at which secondary consolidation begins.

$C_{\alpha\varepsilon}$ = Coefficient of secondary consolidation in terms of strain.

Reference should be made to Geospec 3 (GEO, 2017b) for the relevant laboratory procedures of determining $C_{\alpha\varepsilon}$.

In the case of a single load application, t_0 may be set at the start of loading. In a reclamation, however, t_0 is not obvious because the fill is usually applied gradually but t_0 may be taken as the time at which the load has reached a certain proportion, typically about 90 or 95%, of the final loading (Carter, 1996). For t_1, it may be taken as the time when about 90 to 95% of the primary consolidation has taken place.

Residual settlement due to secondary consolidation

In theory, secondary consolidation continues indefinitely at an ever-decreasing rate. For the purpose of calculating residual settlement, a period of 50 years may be adopted to give the ultimate value. Hence, from Equation 4.5, the residual settlement of a soil layer due to secondary consolidation after a given time t, $S_{s,t(residual)}$, is given by:

$$\begin{aligned} S_{s,t(residual)} &= C_{\alpha\varepsilon} H[\log(\frac{50-t_0}{t_1-t_0}) - \log(\frac{t-t_0}{t_1-t_0})] \\ &= C_{\alpha\varepsilon} H \log(\frac{50-t_0}{t-t_0}) \end{aligned} \qquad (4.6)$$

where t = Time measured from the point at which filling commences (year).

4.4 Assessment of Settlement within Granular Fill

This section describes the principles of settlement analysis for granular fill.

The permeability of granular fill is usually very high compared to the clayey sub-soil. Hence, the primary consolidation will be complete very soon after fill placement and will not contribute to the residual settlement in principle.

The creep settlement of granular fill, however, will take place continuously and may be modelled as a linear relationship with log time by the following formula (Sowers et al, 1965):

$$S_c = H\alpha \log(\frac{t_2}{t_1}) \qquad (4.7)$$

where S_c = Creep settlement of the fill which occurs between times t_1 and t_2 (measured from the time when half of fill placement is completed).

H = Fill thickness.
α = Logarithmic creep compression rate (%).

Residual settlement of fill due to creep

From Equation 4.7, and adopting a cut-off time value of 50 years, the residual settlement of filling materials due to creep after a given time t, $S_{c,t(residual)}$, is given by:

$$S_{c,t(residual)} = H\alpha \log\left(\frac{50 - \frac{t_c}{2}}{t - \frac{t_c}{2}}\right) \tag{4.8}$$

where t = Time measured from the point at which filling commences (year).
t_c = Time for completing fill placement or the construction period (year).

Information on the magnitude of α is limited. It may range from about 1% to 2% for granular fill. The value of α is also sensitive to fill treatment and may reduce to 0.5% to 1% after fill treatment.

4.5 Calculation of Residual Settlement

From Equation 4.3 (without preloading) or Equation 4.4 (with preloading), and together with Equations 4.6 and 4.8, the residual settlement of a reclamation due to the sub-soil and the fill at time t, $S_{residual,t}$, is:

$$S_{residual,t} = S_{p,t(residual)} + S_{s,t(residual)} + S_{c,t(residual)} \tag{4.9}$$

If the soil properties are variable over the depth, the calculation should be broken down into a number of layers, each of which should represent the properties of the soil at the respective depth. The consolidation settlement of granular fill may be ignored because it is generally completed within the reclamation construction period.

Where it is known that development will take place immediately after the completion of reclamation, it is common to limit the residual settlement at the time of completion of the reclamation to less than 500 mm, and 300 mm for areas designated for roads and drains. If the development schedule has not been determined during the design stage, the designer should liaise with relevant government departments or land developers and determine a suitable time interval after completion of reclamation for calculating the target residual settlement. In any case, the designer should check with those bodies to see if there are any special settlement requirements.

For a typical reclamation in Hong Kong where the seabed is underlain by marine and alluvial clay, the following information may assist the designer to assess the residual settlement:

- If the reclamation is near the shore where the thickness of the clay layer is about 5 to 8 m, it is usually practical to limit the residual settlement arising from primary consolidation to 50 mm for the drained method with preloading and a reasonably tight construction programme.

- If the reclamation extends to deep waters, with clay layer of about 20 m or more, residual settlement of 250 mm can generally be achieved. The amount can be further reduced by over-consolidation with additional surcharge, but the cost will be higher and the construction period will be longer. Over-consolidation will also reduce the secondary consolidation, particularly for soft mud.

- The residual settlement due to creep of granular fill, with thickness of 10 to 15 m in a typical reclamation, may be in the order of a few hundred millimetres. Treatment of the fill by the methods mentioned previously may be necessary to achieve the target residual settlement.

The thickness and properties of the soil, the period of fill placement and fill treatment vary across a reclamation site. Therefore, it is necessary to estimate the residual settlement in different portions of the reclamation to account for such variability. Where the presence of mud lenses is suspected in the underlying strata, additional ground investigation should be carried out to identify their locations and to determine relevant soil parameters for settlement analysis.

Where the designer considers that resiltation of clayey or silty particles of the dredged soil is a problem at a particular site, its effect on settlement should be assessed.

The amount of residual settlement estimated in the design stage should be verified by field monitoring data (see Chapter 5 of this part of the Manual).

The calculation of residual settlement for a reclamation is subject to many variables from ground conditions to analytical methods and construction processes. Different practitioners may come up with a different value at the same location due to different assumptions. An appropriate degree of accuracy of the calculated settlement should be indicated in the design reports and technical correspondence.

5. RECLAMATION MONITORING

5.1 General

Instrument monitoring of reclamation provides data for assessing the adequacy of both the sub-soil and reclamation fill with respect to stability and settlement. This chapter gives general guidance on the planning of instrument monitoring for a reclamation scheme and outlines the functions of some typical monitoring instruments. Methods of interpreting monitoring data for stability control during fill placement and verification of settlement calculated in the design stage are also discussed.

5.2 Monitoring Instruments

Instrument monitoring in a reclamation site is usually concerned with measuring the following three basic parameters at different locations and depths in the reclamation:

- Pore water pressure – by piezometers.
- Settlement – by settlement plate, extensometers or surface markers.
- Lateral deformation – by inclinometers.

Techniques for pore water pressure, ground settlement and lateral movement measurement have improved over the years. Instruments capable of measuring the continuous distribution of vertical and lateral movement with high precision are now available in the market. Reference should be made to the specifications of respective manufacturers for different monitoring instruments and specialist advice should be sought where necessary before specifying the monitoring instruments in a reclamation project.

5.2.1 Pore Water Pressure Measurement

It is essential to monitor the pore pressures in a drained reclamation to ensure that the excess pore pressure generated by fill placement are in line with predictions, and to confirm that the vertical drains are functioning properly.

Pore water pressure is monitored using piezometers. For further details, Chapter 20 of Geoguide 2 (GEO, 2017a) should be referred to. Vibrating wire piezometers are usually adopted for monitoring pore pressures in reclamation. These have a rapid response and are relatively simple to set up.

For monitoring of pore pressure during fill placement, the effect of tidal variation on the piezometer readings should be checked in order to obtain the excess pore water pressure at a given time and location. A piezometer may be installed outside the reclamation to provide simultaneous tidal data for excluding the tidal component on piezometer data taken within the reclamation (see Figure 9). The fill level at the piezometer location should also be recorded so that the measured excess pore water pressure can be compared with the total vertical stress increase due to fill placement.

5.2.2 Settlement

Settlement is an important monitoring parameter for both drained and dredged reclamation. The magnitude and rate of settlement may be measured by means of settlement plates or extensometers. Settlement plates, installed either on the seabed, within the fill or at the surface of the reclamation, measure the settlement by comparing the levels of the plates taken at different times, with a fixed datum level. If settlement characteristics of individual sub-soil layers are required, extensometers are more suitable. The settlement of individual sub-soil layers can be determined by detecting the relative change in the vertical positions of the steel or magnetic ring targets in the extensometers by a probe.

5.2.3 Lateral Deformation

Lateral deformation can be measured by inclinometers. Lateral deformation data are particularly useful during fill placement in a drained reclamation, as analysis of the data trend can reveal early signs of instability in the sub-soil.

In monitoring reclamation, an inclinometer is usually combined with an extensometer to form an inclinometer-extensometer composite installed in a pre-drilled hole. The fill level at and around the instruments should be recorded simultaneously when taking the readings.

5.3 Monitoring for Drained Reclamation

Reliable monitoring data is necessary for:

- Monitoring of stability for drained reclamation.
- Verification of predicted settlement.

5.3.1 Instrument Monitoring Locations

With prudent planning, instrumentation can be installed before the start of fill placement so that measurements can be taken prior to and during the course of filling. Instrumentation should be concentrated at critical areas such as those where the marine mud is thick or particularly weak. Since lateral deformation of the marine mud, if any, is generally more prominent at the edge of reclamation, some instrumentation should be placed near the leading edge of the reclamation where possible in order to effectively monitor the stability.

The location of the instruments may need to be refined after finalizing the working sequence with the reclamation contractor. Installation of monitoring instruments prior to the commencement of reclamation may cause obstruction to the movement and operation of marine plant on the site. There is also no point in putting in instrumentation early if the area will not be reclaimed until a late stage. Therefore, the locations of the monitoring points should only be finalized when the reclamation sequence and plant operation are known.

If the instruments are intended to be placed at locations of future surcharge mounds, the instruments will need to be installed in a manner that allows later vertical extension. In general, instruments should not be installed in future building areas or other areas where piling may be required, as the instruments are likely be destroyed. The ideal locations are in areas zoned for future roads, footpaths, cycle tracks, amenity areas and open spaces.

Instruments may involve underwater installation and may need protection to withstand the marine environment and fill placement operations. Figure 10 illustrates the set-up of monitoring instrumentation installed by marine plant prior to the commencement of fill placement and by land plant after the reclamation is filled up to above water level. For instruments installed prior to commencement of filling, protection can be achieved by housing all the instruments of each cluster in a rigid sleeve above the seabed. Fixed temporary staging should be erected to support and protect the sleeve. The temporary staging should be rigid and strong enough to ensure that the sleeve can stand above the seabed in a stable manner. Due care should be exercised to avoid damaging the instrumentation while placing fill in the vicinity. For instance, grabbing instead of bottom dumping may be adopted adjacent to the instruments. If installation of instruments by marine plant prior to commencement of fill placement is not feasible, instruments should be installed immediately after the reclamation is filled up to above water level so that monitoring can be started at the earliest opportunity.

5.3.2 Monitoring of Stability During Fill Placement

This section discusses the methods of plotting the collected monitoring data to assist in the assessment of the interim stability of the reclamation process. However, other site observations will also be useful indicators of the reclamation stability such as:

- Tension cracks appearing at the top and side slope of the reclamation.
  Settlement at the centre of reclamation increasing rapidly.
- Sounding survey results indicating up and down curve profiles of the seabed near the edge of the reclamation.
- Filling rate increasing suddenly.
- Fill being stockpiled on the site excessively.

Monitoring data should be taken at least daily and given to the engineer immediately to enable any anomalous behaviour to be discovered as early as possible.

(1) u-σ Plot

Excess pore water pressure (u) can be plotted against the vertical stress (σ) due to the reclamation fill (γH) (see Figure 11). For a stable reclamation process, the u-σ Plot should show an increase in pore water pressure which is either smaller than or the same as the increase in vertical stress in the course of fill placement. If the rise in pore water pressure is much faster than the rise in vertical stress, action must be taken to avoid failure.

(2) s- δ_H Plot

Settlements of sub-soils at about the centre of the reclamation area (s) can be plotted against the lateral deformation in the mud layer near the edge of the fill (δ_H). Experience reveals that when δ_H /s increases rapidly, it is an indication of imminent failure of the mud layer (Ye et al, 1997). The line s-δ_H is straight and inclined at an angle to the s-axis when the vertical stress is small (see Figure 12). However, when failure is imminent, δ_H will increase at a faster rate than s and the straight line will curve towards the δ_H - axis.

(3) $\Delta h/\Delta\delta_H$ - h Plot

Figure 13 shows the typical relationship between the height of filling (h), time (t) and lateral deformation (δ_H). During the filling operations, the reclamation level is raised from h to $h+\Delta h$, and the change in lateral deformation $\Delta\delta_H$ is measured after time Δt. According to

Ye et al (1997), the data set Δh, $\Delta\delta_H$ and h can be used to estimate the maximum allowable height of fill. There is a consistent relationship between $\Delta h/\Delta\delta_H$ and h as shown in Figure 13. When h is small, $\Delta h/\Delta\delta_H$ is large. As h gets larger, the relationship between $\Delta h/\Delta\delta_H$ and h will tend to become linear. If this straight portion is projected towards the h-axis, the intercept will give the predicted maximum height of filling. The height of fill should not exceed this value until after consolidation of the mud layer (checked by monitoring pore pressures).

5.3.3 Verification of Settlement Based on Field Monitoring Data

The principle of verification of settlement involves back-analysis of field monitoring data. This can be carried out by fitting different values of coefficient of consolidation (vertical and horizontal) into the consolidation equations and comparing the resultant settlement-time relationships with the actual settlement-time curve plotted from field settlement data. Both Terzaghi's one-dimensional consolidation theory and Barron's radial consolidation theory for vertical and horizontal drainage respectively can be applied in the back-analysis (see Appendix B). The process of curve fitting is a trial and error process which can be speeded up with the use of computer programmes incorporating the appropriate consolidation equations and assumptions. Wai and Lam (2001) may be referred to for more details of back-analysis by curve fitting.

Alternatively, a method developed by Asaoka (1978) also enables utilization of the settlement monitoring data, through simple graphical means, to verify the settlement due to primary consolidation calculated in the design stage. The principles and details of application of the method are given in Appendix C. The results provide additional information for the designer to assess the settlement condition of the reclamation.

Asaoka's method is based on observational settlement prediction. Field settlement data measured by settlement plates at the seabed or, if not practicable, at the surface of the drainage blanket, are plotted in accordance with the method given in Appendix C. The plotted results give an estimate of total primary consolidation settlement of the soil beneath the seabed, which can then be compared with the total primary consolidation settlement predicted at the design stage. The residual settlement can also be re-assessed with reference to the ultimate primary consolidation settlement estimated from Asaoka's Method and the field settlement data measured at a particular time, say, at the time of completion of filling works or handing over the site.

Asaoka's method has the advantage that it is simple to use. However, it relies very much on the availability of frequent monitoring data. For a reclamation project with a fill placement duration of about two to three years, seabed settlement monitoring data at a frequency of about one month is required.

Both the back-analysis method and the Asaoka's method can be applied to counter check the results of each method.

5.4 Monitoring for Dredged Reclamation

The principles described in Section 5.3 above also apply to dredged reclamation. Settlement is usually the main parameter that is monitored in a fully dredged reclamation in order to confirm prediction of settlement. For a partially dredged reclamation, monitoring instruments similar to those in a drained reclamation may be set up to measure the pore water pressure, settlement and lateral deformation. However, fewer instruments may be required, depending on the type and thickness of the remaining sub-soils. Data interpretation will be similar to that for a drained reclamation.

Where the dredged reclamation is carried out adjacent to existing developments or structures, monitoring instruments should be installed in the existing ground to monitor pore water pressures and soil movements, and hence stability of these developments or structures.

6. MISCELLANEOUS DESIGN ASPECTS

6.1 General

Brief comments on particular aspects of design related to piling, culvert foundations and other miscellaneous structures and facilities in a reclamation are given below.

6.2 Piling

Obstructions to piling activities at reclamation sites can result in serious cost overruns and programme delays. The largest rock or boulder encountered when driving piles must be breakable by impact or be capable of being displaced. For the latter, there must be sufficient voids in the fill to cope with the pile volume and allow the displacement. Experience has shown that 250 mm is the approximate upper size limit for boulders within the fill to avoid major problems with the installation of common pile types such as driven concrete piles, steel H-piles, steel tubular piles and bored piles.

Permanent steel liners are usually necessary for bored piles to avoid necking, particularly in crushed rock fill or where pockets of unconsolidated marine deposits are suspected. Special shoes or reinforced sections or points are normally required for driven piles, particularly in public fill or crushed rock fill.

6.3 Culvert Foundations

The type of foundation to be used for a culvert extension across the new reclamation will depend on the method of reclamation used and the amount of the residual settlement. To reduce residual settlement, surcharging may be carried out with surplus fill over a strip two to three times the width of the culvert and about five to six metres high until settlement has stabilized.

Where the marine deposits have been fully removed or where the residual settlement has effectively been completed before culvert construction commences, a nominal crushed rock or rubble foundation layer will usually be acceptable.

Where the residual settlement is not effectively complete or where variable thicknesses of marine deposits are suspected, surcharge preloading and fill densification measures can reduce settlement problems. However, it is recommended that special measures are taken to allow for some differential settlement at the culvert joints. The use of piled foundations may be considered where differential settlement problems are expected to be particularly severe, but the initial cost will be relatively expensive, and can result in other forms of settlement problems, as outlined below.

Normally, culvert outfalls are formed in the seawalls using special concrete outfall blocks. The difference in settlement between the seawalls and the reclamation will cause differential movement at the junction with a piled culvert. In addition, differential settlement will also occur along each side of a piled culvert, as the adjacent reclamation continues to settle.

6.4 Structures and Facilities

Settlement, in particular differential settlement, of reclamation may adversely affect the structural integrity and serviceability of structures and facilities to be built on the reclaimed land. General design aspects of pavements, structures, pipes and ducts, constructed on a reclamation are described in the following paragraphs.

For carriageways and pedestrian walkways constructed on newly reclaimed land, differential settlement may cause non-uniform deformations and hence affect the serviceability. These effects can be mitigated by using flexible pavement that allows future overlay.

To avoid the possibility of differential settlement, buildings constructed on reclaimed land are usually supported on a piled foundation. The possible effects of negative skin friction on the pile due to settlement must be taken into account in the design. The magnitude of the negative skin friction at a particular site will depend on the pile characteristics, fill characteristics, pile movement and the settlement since completion of the pile. Other types of foundation, such as a raft or floating foundation, may be considered for low-rise buildings.

For underground drainage and sewerage pipes built in reclamation areas, the effect of differential settlement may be mitigated by the use of flexible joints, particularly for the connections between the pipes or ducts entering buildings. The adoption of several short pipe lengths and flexible joints adjacent to buildings will allow the pipes to accommodate quite severe differential settlement. The design should ensure that adequate fall is maintained in gravity sewers after settlement is complete.

When designing structures and facilities to be sited on a new reclamation, reference should be made both to the settlement calculation and the settlement data recorded during the course of reclamation. Additional ground investigations may also need to be carried out after the completion of reclamation to obtain the necessary soil parameters for the design. Close consultation and liaison between the design office, client and maintenance authorities are necessary throughout various stages of the project.

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FIGURES

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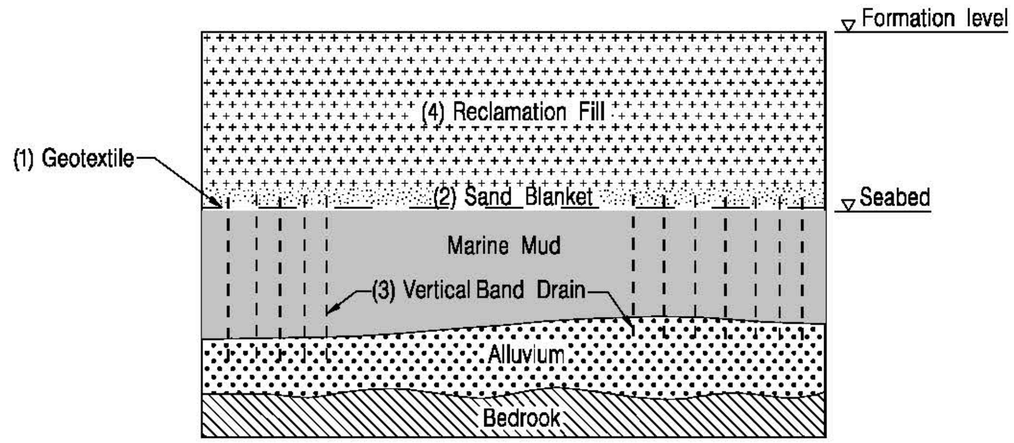


Notes:

1. Laying of geotextile on the seabed.
2. Deposition of blanket layer.
3. Installation of vertical drains by marine plant.
4. Controlled thin-layer placement of reclamation fill.

Figure 1 – Sequence of Drained Reclamation

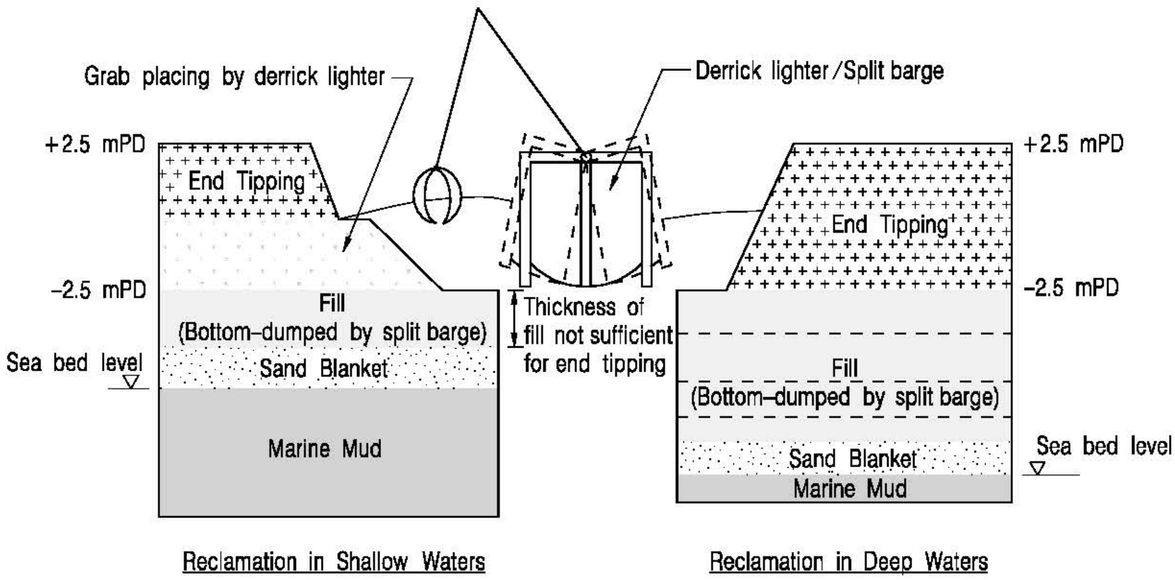
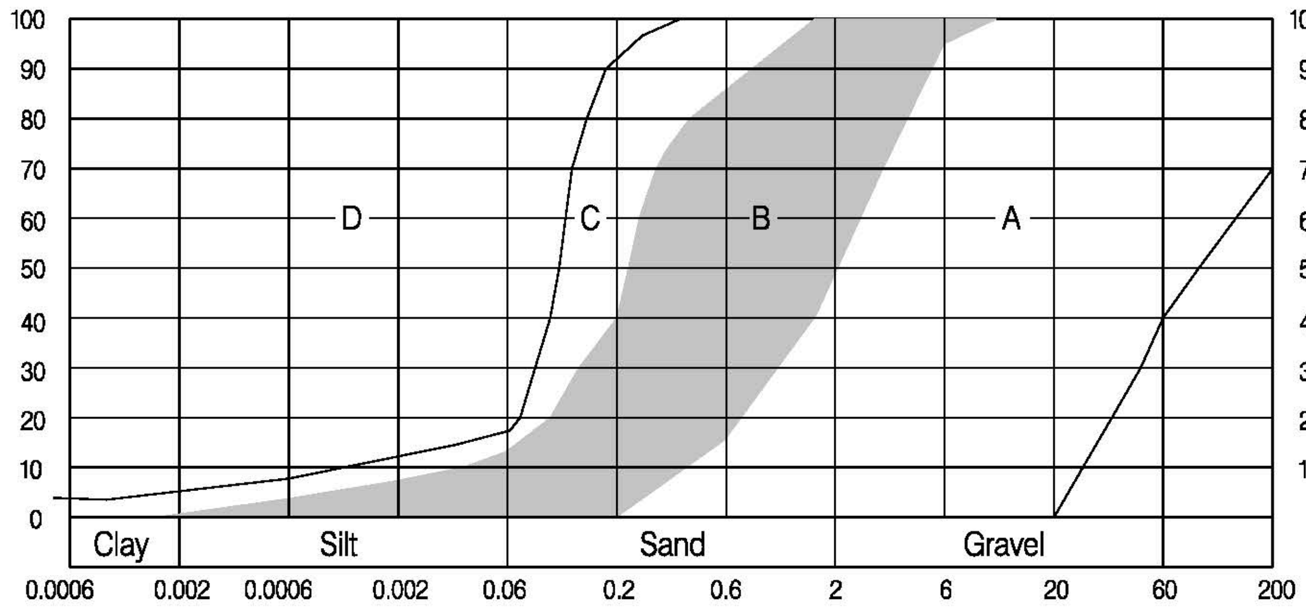


Figure 2 – Reclamation in Shallow Waters and Deep Waters



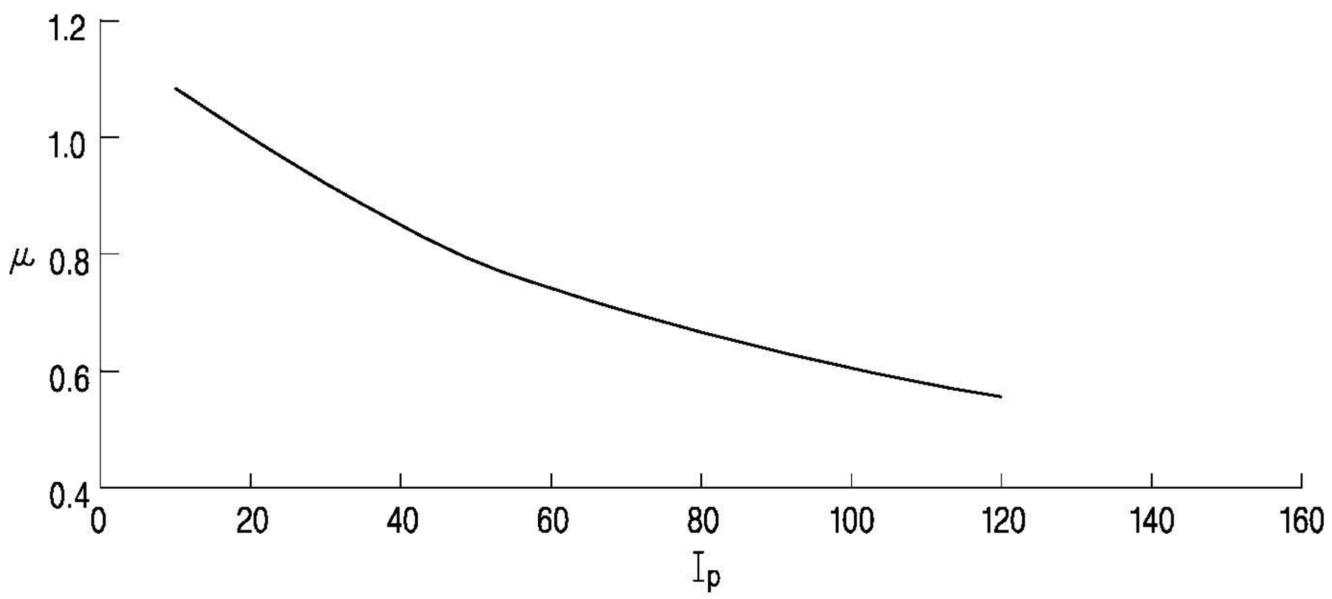
Zone A: The soils of this zone are very well compactable.
The rightmost borderline in the graph indicates an empirically found limit where the amount of cobbles and boulders prevents compaction because the vibroprobe cannot reach the compaction depth.

Zone B: The soils in this zone are ideally suited for vibrocompaction. They have a fines content of less than 12%. As for the soils of zone A, the in-situ soil flows towards the vibroprobe during compaction, so that no backfill has to be added from the top, providing the resulting settlements of the ground are tolerable. Depending on the initial density and the required densification, settlements due to vibrocompaction are between 2% and 15% of the thickness of the treated layers.

Zone C: Zone C is still suitable for vibrocompaction, but the required compaction time is drastically increased compared to zone B. This happens because the surplus water does not drain fast enough from the compacting soil.
A compaction is only possible by adding backfill from the surface, since the in situ soil does not flow by itself towards the vibroprobe.

Zone D: Soils of this zone are not compactable.

Figure 3 – Soils Suitable for Vibrocompaction



Ref. : Bjerrum (1972)

Figure 4 – Relationship between Correction Factor μ for Field Vane Shear Strength and Plasticity Index

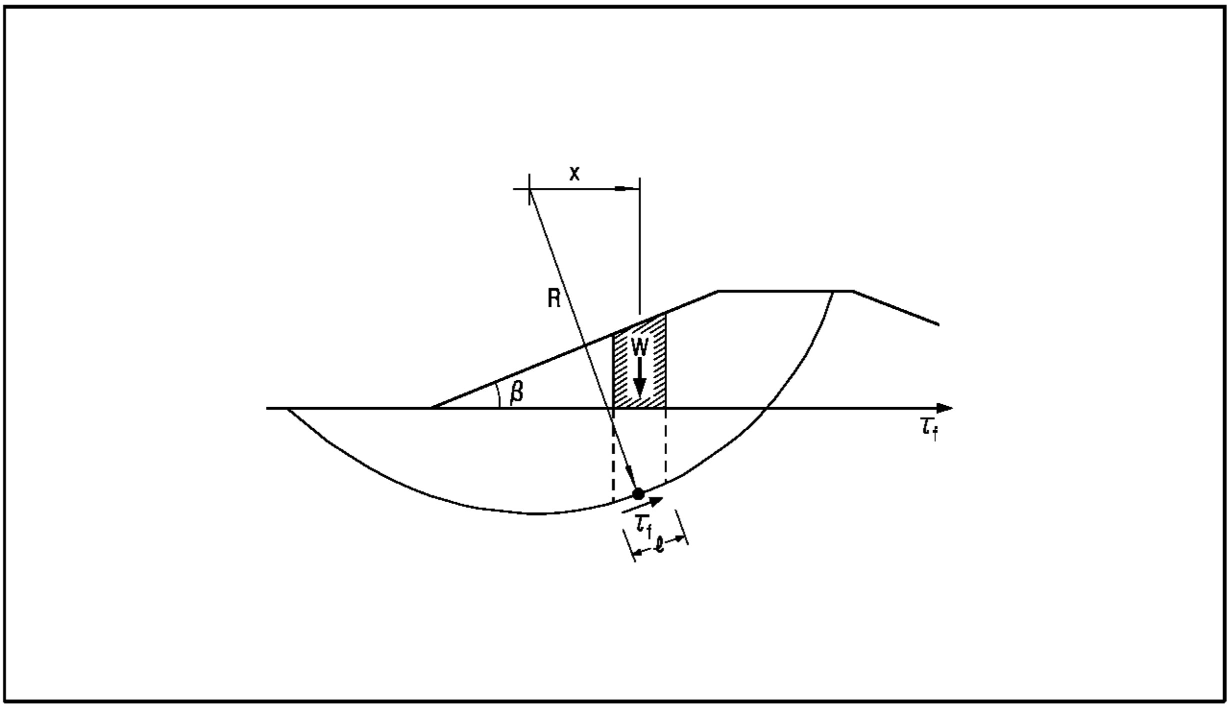


Figure 5 – Method of Calculation of Stability of an Embankment on Clay

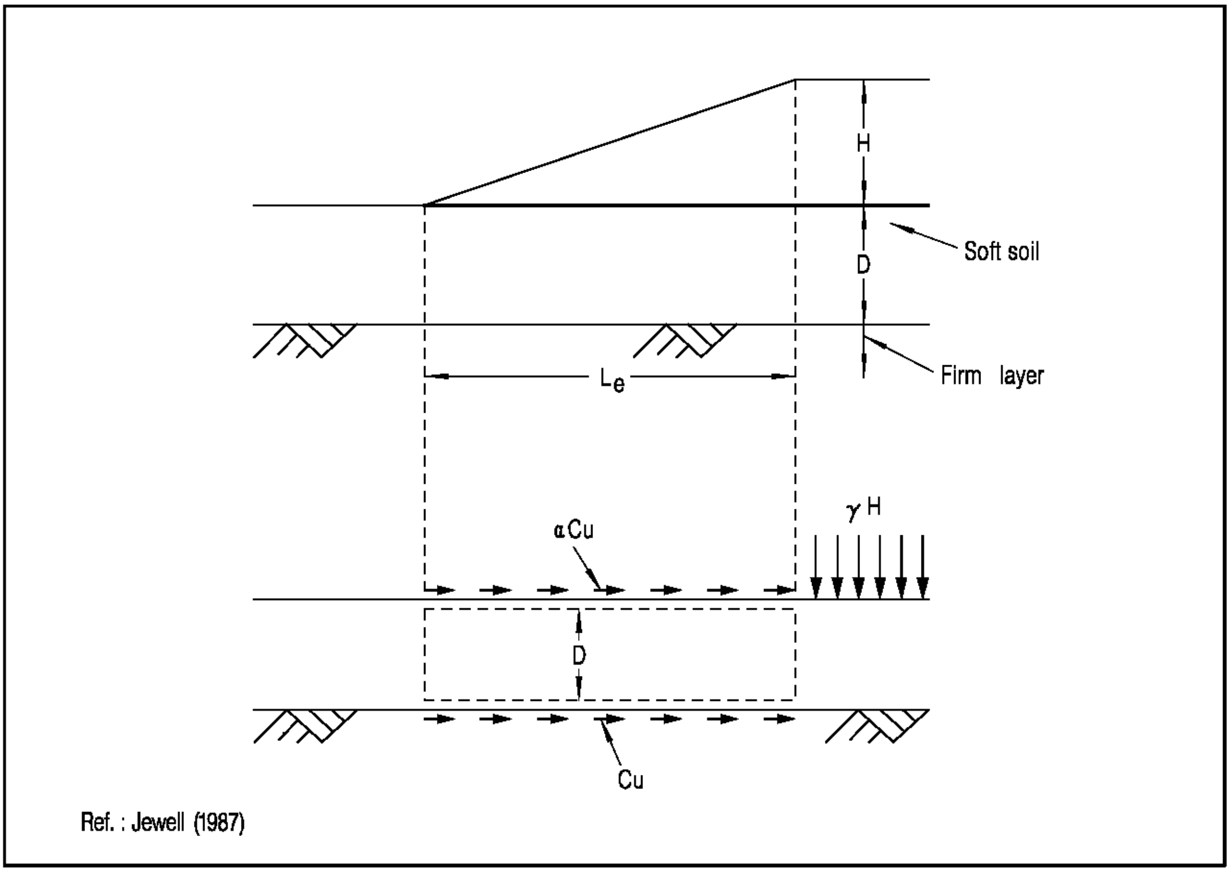


Figure 6 – Foundation Soil of Limited Depth and Uniform Strength

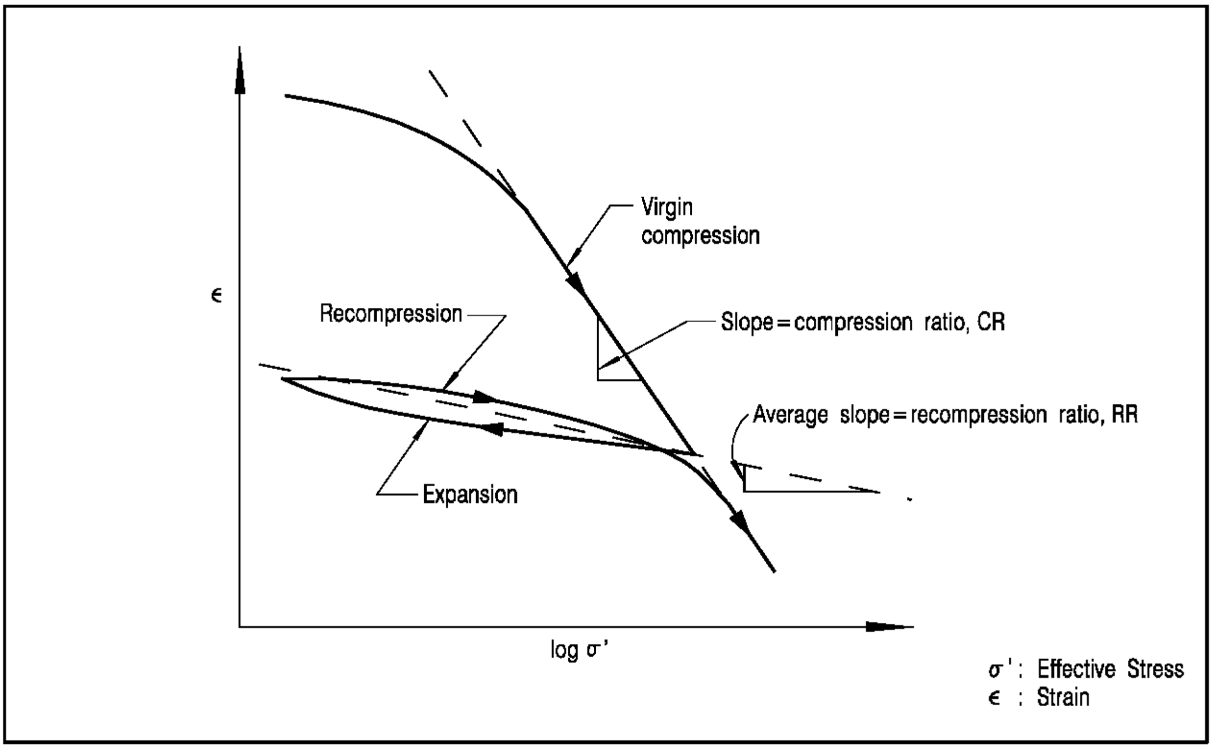
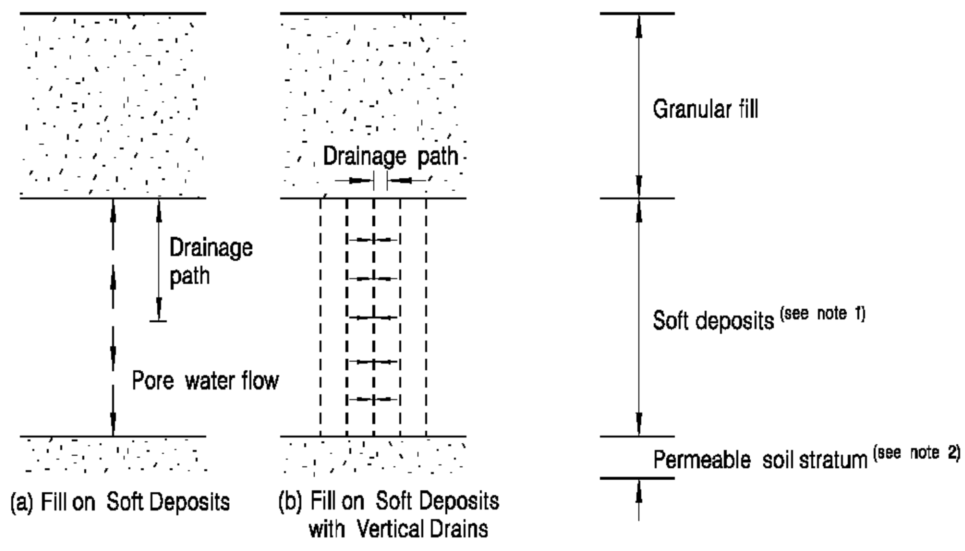


Figure 7 – Definition of Compressibility Parameters

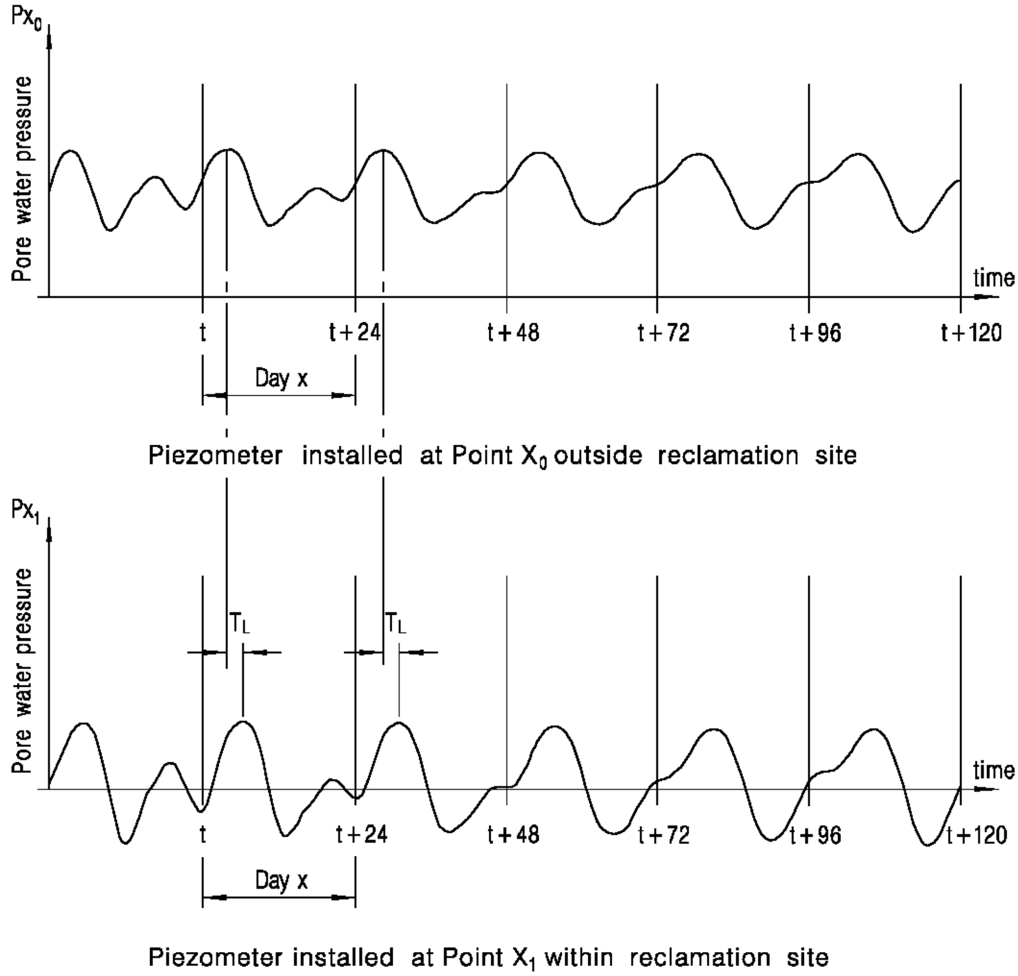


Notes:

1. Soft deposits include marine and alluvial deposits.

2. Soft deposits may lie within impermeable strata such as stiff alluvial clay.

Figure 8 – Shortening of Drainage Path by Vertical Drains



Average excess pore water pressure at Point X_1 on Day x	=	Average of the pore water pressure recorded by piezometer at x_1 between time $(t+T_L)$ and $(t+24+T_L)$ – Average of the pore water pressure recorded by piezometer at x_0 between time t and $(t+24)$

Legend:

Px_0 = Pore water pressure recorded by piezometer at Point x_0 outside reclamation.

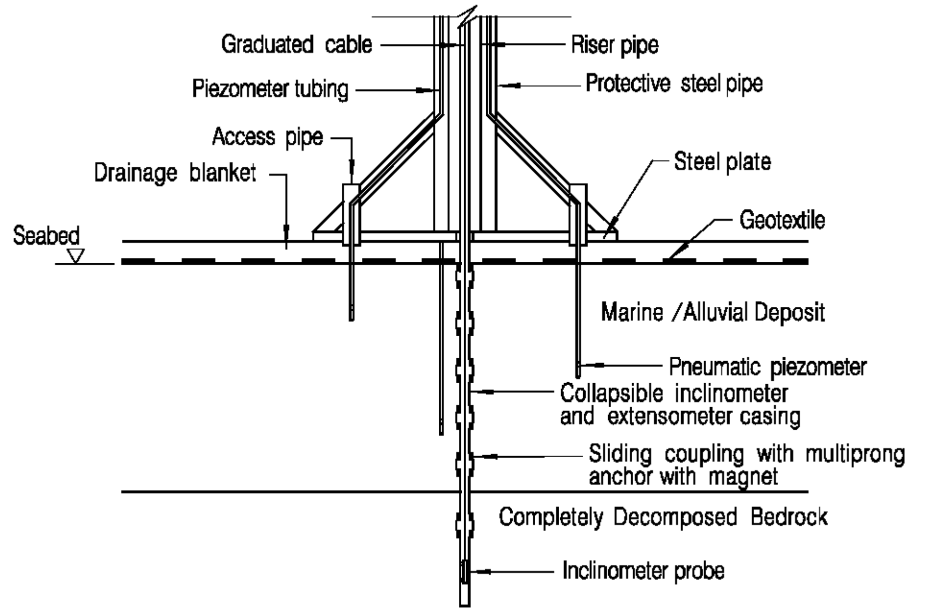
Px_1 = Pore water pressure recorded by piezometer at Point x_1 within reclamation.

T_L = Time lap of response between piezometers at Points x_0 and x_1.

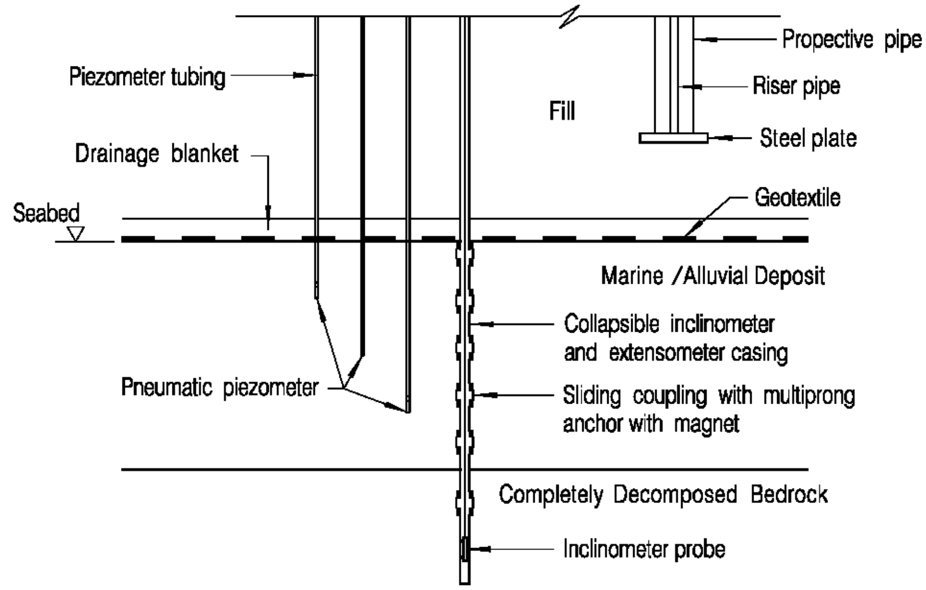
Notes:

1. Automatic electronic device should be used to facilitate recording of pressure data.
2. Pressure reading should be logged continuously with time.
3. Piezometer outside the reclamation is installed for the purpose of eliminating the tidal effect on the piezometer reading taken inside the reclamation.

Figure 9 – Tidal Effect on Pore Water Pressure Measurement



(a) Installed by Marine Plant (before Commencement of Filling)



(b) Installed by Land Plant (after Filling to Above Water Level)

Figure 10 – Settlement Monitoring Instrument Cluster

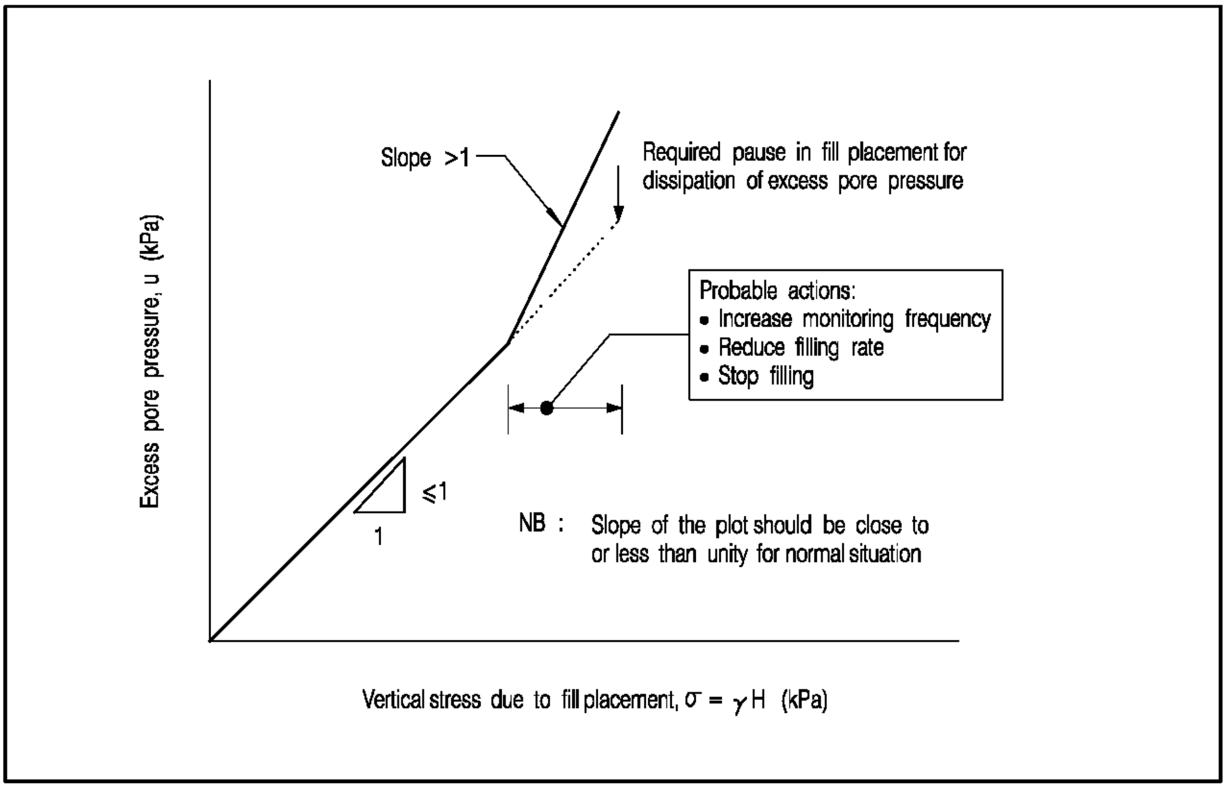


Figure 11 – u–σ Plot

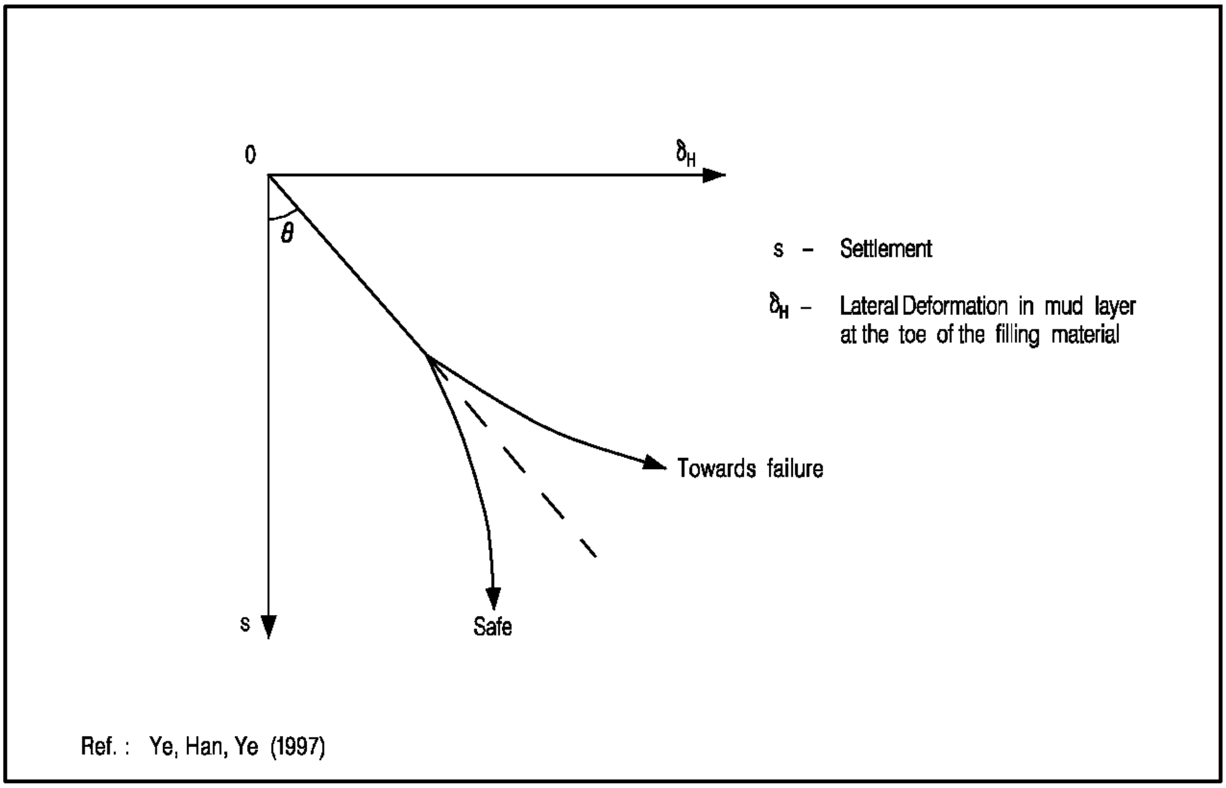
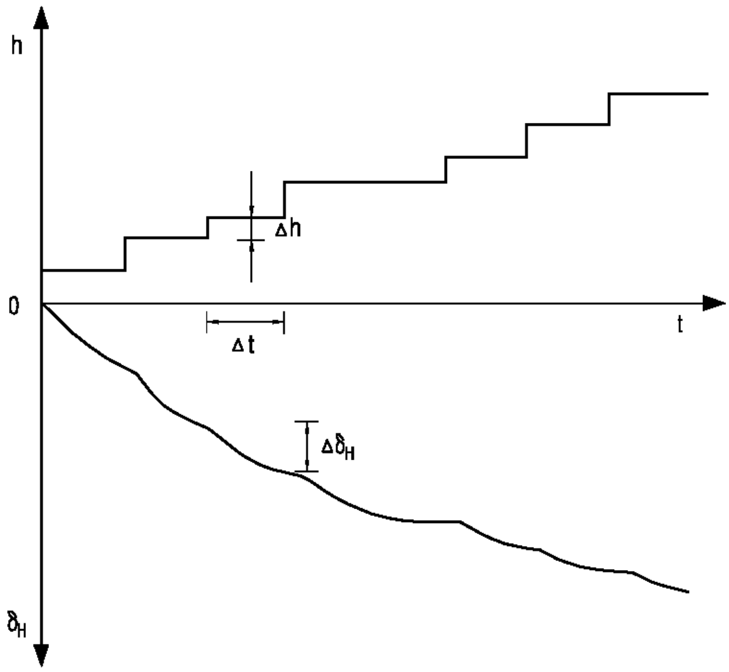
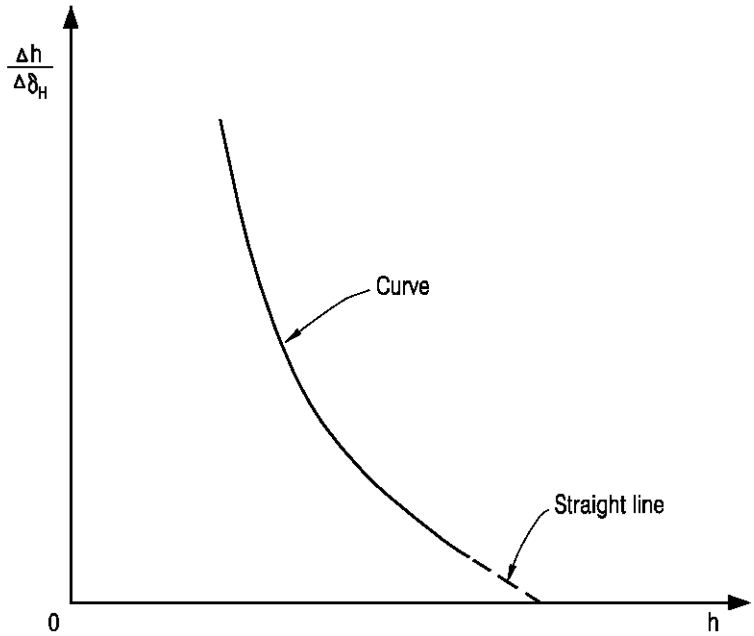


Figure 12 – s–δ_H Plot



(a) δ_H–h–time Plot



(b) $\frac{\Delta h}{\Delta \delta_H}$ –h Plot

Ref. : Ye, Han, Ye (1997)

Figure 13 – Typical Relationship between Height of Filling (h), Time (t) and Lateral Deformation (δ_H)

APPENDIX A

PREFABRICATED BAND DRAIN INFORMATION

APPENDIX A PREFABRICATED BAND DRAIN INFORMATION

Specifications for Prefabricated Vertical Band Drain

Component	Property	Test Method	Requirements
Prefabricated Band Drains	Width	-	100 mm ± 5 mm
	Discharge capacity q_w under straight condition of flow (for confining pressure at 240 kPa and hydraulic gradient at 0.5)	ASTM D4716-87 (See Note 2)	> 55 x 10^{-6} m^3/s
	Tensile strength	ASTM D4632-91 (See Notes 3 and 4)	> 1,000 N
	Elongations at 1 kN	ASTM D4632-91	< 10%
Filter	Apparent opening size (AOS = O_{95})	ASTM D4751-93	< 90 µm
	Permittivity	ASTM D4491-92	> 0.2 s^{-1}

Notes:

(1) All testing methods refer to American Society for Testing and Materials (ASTM).

(2) (a) $q_w = q_i / i$ where q_i is the flow rate at hydraulic gradient i.
(b) Soft neoprene should be used to simulate soft clays.
(c) De-aired water should be used in the test.

(3) None of the following items shall break before reaching the stipulated tensile strength.
- Drain core.
- Drain filter fabric.
- Seam of the filter fabric.

(4) The ASTM D4632-91 Test shall be carried out using full width jaws on full width of prefabricated vertical band drains.

(5) Frequency of quality control test shall be one test for 50,000 m length of drain installed.

List of ASTM Standards

D4491-92: Standard Test Methods for Water Permeability of Geotextiles by Permittivity

D4632-91: Standard Test Method for Breaking Load and Elongation of Geotextiles (Grab Method)

D4716-87: Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products

D4571-93: Standard Test Method for Determining Apparent Opening Size of a Geotextile

APPENDIX B

ASSESSMENT OF DEGREE OF CONSOLIDATION

CONTENTS

APPENDIX B ASSESSMENT OF DEGREE OF CONSOLIDATION

B.1 General

This appendix gives guidance on the principles of calculating the average degree of consolidation for a homogeneous soil stratum with or without vertical drains. When a vertical load is applied, an initial uniform excess pore pressure is assumed generated instantaneously throughout the soil stratum. This excess pore pressure will dissipate gradually or almost instantaneously, depending on the permeability of the soil and the drainage conditions at the boundaries of the stratum, by vertical drainage through the soil to the horizontal boundaries and/or by radial drainage into pre-installed vertical drains. The average degree of consolidation indicates how much of the imposed load is transferred to the effective stress in the soil (with 100% meaning full transfer), defined by:

$$U_t = \frac{u_i - u_t}{u_i} \times 100\% \qquad \text{(B1)}$$

where U_t = Average degree of consolidation of a homogeneous soil stratum at a particular time.

u_i = Initial excess pore pressure upon application of a vertical load.

u_t = Average excess pore pressure at a particular time.

B.2 Consolidation without Vertical Drains

Terzaghi's theory of one-dimensional consolidation predicts the excess pore pressure under vertical drainage alone. Based on Equation B1, Terzaghi's theory gives the following term for the average degree of consolidation due to vertical drainage:

$$U_v = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp(-M^2 T_v) \qquad \text{(B2)}$$

where U_v = Average degree of consolidation due to vertical drainage alone.

M = $\frac{\pi}{2}(2m+1)$

T_v = $\frac{c_v t}{d^2}$

d = Length of longest drainage path.

t = Time from load application.

c_v = Coefficient of consolidation due to vertical drainage, which can be obtained from laboratory oedometer tests.

The variation of the degree of consolidation with depth is given by:

$$U_{v,z} = 1 - \sum_{m=0}^{\infty} \frac{2}{M} \sin\left(\frac{Mz}{H_{dr}}\right) \exp(-M^2 T_v) \quad \text{(B2a)}$$

where $U_{v,z}$ = Degree of consolidation due to vertical drainage alone at depth z.

H_{dr} = Length of drainage path.

Reference may also be made to Lee et al (1992) for more accurate assessment of the degree of consolidation for layered deposits.

B.3 Consolidation with Vertical Drains

B.3.1 Consolidation due to Horizontal Drainage

The classical solution is analogous to that described in Section B.2 but flow is horizontal and radially inwards towards each drain. Barron (1948) arrived at the solutions for the excess pore pressure at any radial distance from the drain and at any time during consolidation. Based on Equation B1 and the assumption of equal vertical strain, the following expression can be established for the average degree of consolidation due to horizontal drainage:

$$U_h = 1 - \exp\left[\frac{-8T_h}{F(n)}\right] \quad \text{(B3)}$$

where T_h = $\frac{c_h t}{D^2}$

c_h = Coefficient of horizontal consolidation.

D = Diameter of equivalent cylinder of soil drained by each vertical drain.

= $1.13 \times$ drain spacing for square grid

= $1.05 \times$ drain spacing for triangular grid

F(n) is a function mainly relating to drain spacing and size, and the extent of soil disturbance due to drain installation (smear effect). It has more than one version as addressed in detail in Barron (1948) and Holtz et al (1991). The basic form of *F(n)* for ideal drains with no smear effect can be expressed as:

$$F(n) = \frac{n^2}{n^2 - 1}\ln n - \frac{3n^2 - 1}{4n^2} \qquad \text{(B4)}$$

where n = D/d'
d' = Drain diameter.
= drain circumference / π

Regarding well resistance, the effect depends on the permeability of the vertical drains and the surrounding undisturbed soil as well as the drain diameter. Different versions of a modified form of Equation B3 that take into account the effect of well resistance (and smear effect) may be found in Hansbo (1981) and Onoue (1992).

B.3.2 Consolidation under Combined Vertical and Horizontal Drainage

The presence of vertical drains does not prevent the vertical drainage of water in the normal way. In reality, both horizontal and vertical drainage take place simultaneously. This can be taken into account using the relationship suggested by Carillo (1942):

$$U_f = U_h + U_v - U_h U_v \qquad \text{(B5)}$$

where U_f is the average degree of consolidation under combined vertical and horizontal drainage.

B.4 References

Barron, R.A. (1948). Consolidation of Fine Grained Soils by Drain Wells. Transactions of the American Society of Civil Engineers, Vol. 113: 718-724.

Carillo, N. (1942). Simple Two and Three Dimensional Cases in the Theory of Consolidation of Soils. Journal of Maths and Physics, Vol. 21 (1).

Hansbo, S. (1981). Consolidation of Fine-grained Soils by Prefabricated Drains. Proceedings of the 10th ICSMFE, Vol. 3: 677-682.

Holtz, R.D., Jamiolkowski, M.B., Lancellotta, R. and Pedroni, R. (1991). Prefabricated Vertical Drains: Design and Performance. CIRIA/Butterworth Heinemann, London/Oxford.

Lee, P.K.K., Xia, K.H. and Cheung, Y.K. (1992). A Study on One-dimensional Consolidation of Layered Systems. International Journal for Numerical and Analytical Methods in Geomechanics, Vol 16, pp 815-831.

Onoue, A. (1992). GEOTECH 92 – Contributions of Dr. Atsuo Onoue to the Workshop on Applied Ground Improvement Techniques. Southeast Asian Geotechnical Society (SEAGS), Asian Institute of Technology, Bangkok, Thailand.

APPENDIX C

VERIFICATION OF SETTLEMENT BY ASAOKA'S METHOD

CONTENTS

APPENDIX C VERIFICATION OF SETTLEMENT BY ASAOKA'S METHOD

C.1 General

This appendix describes the method, namely Asaoka's graphical method, which can be used to verify the primary consolidation settlement using the field monitoring data of settlement.

C.2 Principle of Verification

If a function *s(t)* is given by

$$s(t) = a(1 - be^{-ct}) \qquad \text{where } a,\ b \text{ and } c \text{ are constant, and } c > 0 \qquad \text{(C1)}$$

the discrete values of *s(t)* at equal time intervals of Δt can be characterised by the following recurrence equation:

$$s_i = \beta_0 + \beta_1 s_{i-1} \qquad [\quad s_i \text{ means } s(t_i = i\Delta t) \quad] \qquad \text{(C2)}$$

where $$\beta_1 = e^{-c\Delta t} \qquad \text{(C3)}$$

According to the solutions for both Terzaghi's one dimensional theory of consolidation and Barron's theory of radial consolidation, the primary consolidation settlement at time *t* and *s*(*t*) can be expressed in the form of Equation C1 as below:

Terzaghi:

$$s(t) = s_\infty \left(1 - \frac{8}{\pi^2} e^{-\frac{\pi^2 c_v t}{4H^2}}\right) \qquad \text{(C4)}$$

$$\text{Therefore,} \quad \beta_1 = e^{-\frac{\pi^2 c_v \Delta t}{4H^2}}$$

Barron:

$$s(t) = s_\infty \left(1 - e^{-\frac{8 c_h t}{D^2 F(n)}}\right) \qquad \text{(C5)}$$

$$\text{Therefore,} \quad \beta_1 = e^{-\frac{8 c_h \Delta t}{D^2 F(n)}}$$

The meanings of the constants *H*, *D* and *F*(*n*) are given in Appendix B.

The values of the ultimate primary consolidation settlement, s_∞ , can be readily obtained using the common methods of solving this type of equation. The graphical method suggested by Asaoka (1978) to find the ultimate primary consolidation settlement using field measurement data is illustrated in Figure C1. This is based on the linear relationship between s_{i-1} and s_i given in Equation C2. The intersection of the $s_{i-1} \sim s_i$ line ($s_i = \beta_0 + \beta_1 s_{i-1}$) with another line $s_{i-1} = s_i$ will give the ultimate primary consolidation settlement (s_∞).

In case of multi-stage loading, the line $s_i = \beta_0 + \beta_1 s_{i-1}$ will be moved up as shown in Figure C2. When the settlement is relatively small compared with the thickness of soil layer, the shifted line should be almost parallel to the initial line because β_1 should be independent of the applied load.

C.3 References

Asaoka, A (1978). Observational Procedure of Settlement Prediction. Soils and Foundations, Vol. 18, No. 4, Dec 1978, Japanese Society of Soil Mechanics and Foundation Engineering.

Kwong. J. S. M. (1996). A review of Some Drained Reclamation Works in Hong Kong. Geo. Report No. 63, Geotechnical Engineering Office, Hong Kong, 53p.

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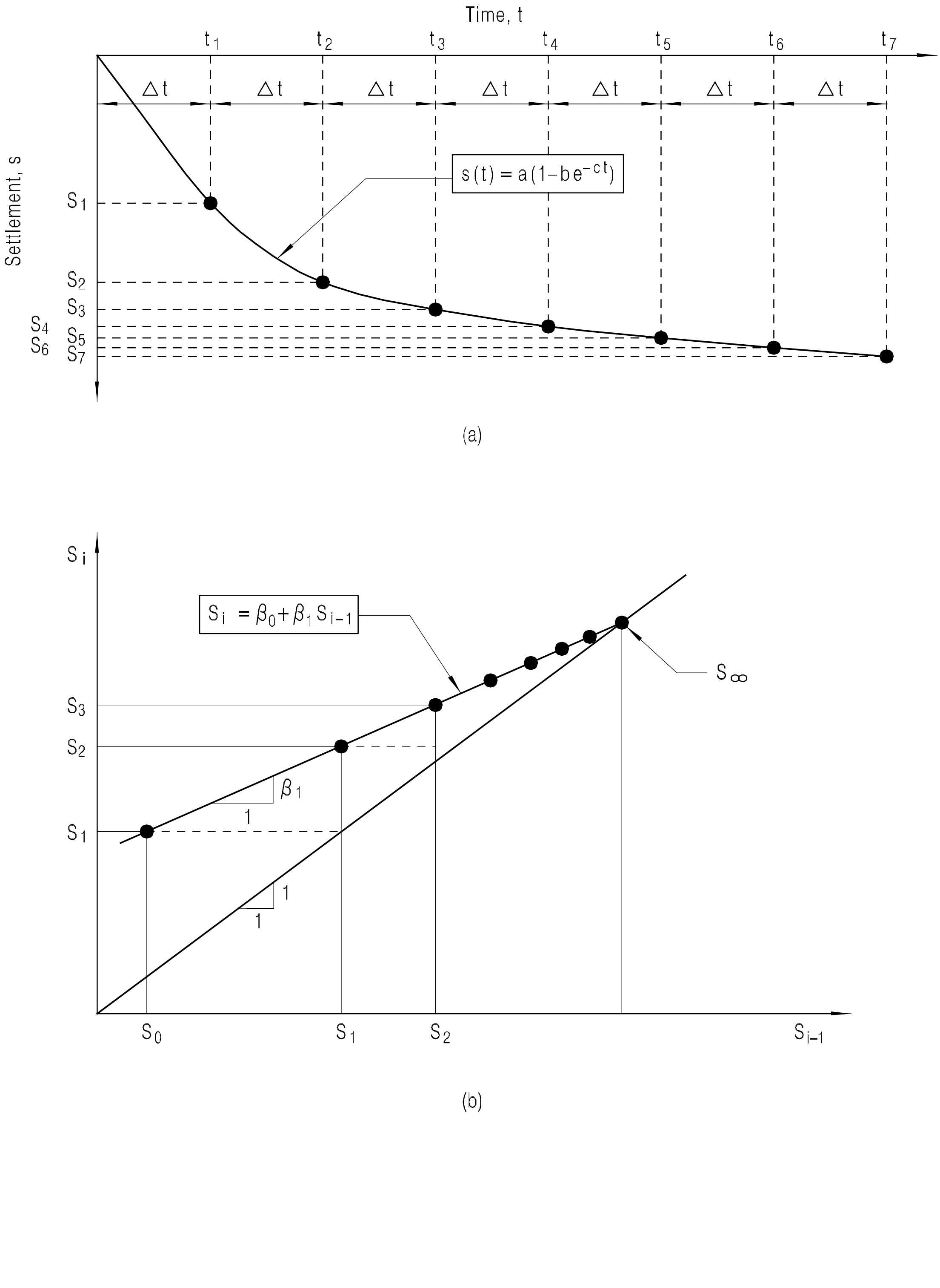


Ref: Kwong (1996)

Figure C1 – Asaoka's Graphical Method of Settlement Prediction

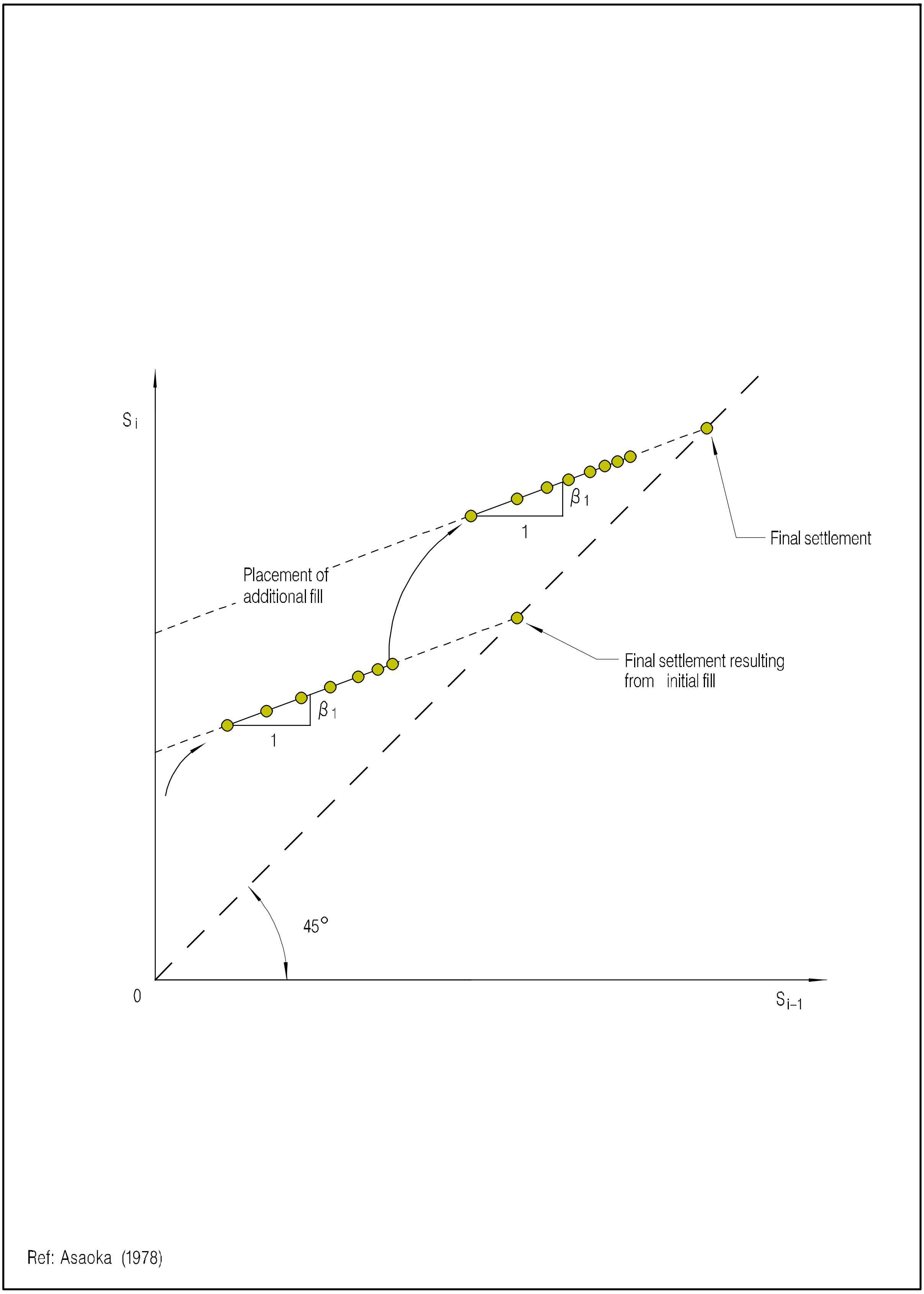


Figure C2 – Asaoka's Graphical Method of Settlement Prediction for Multi-stage Loading

APPENDIX D

WORKED EXAMPLES

CONTENTS

APPENDIX D WORKED EXAMPLES

D.1 Example 1 – Stability Analysis for Leading Edge of Reclamation

Given

1. A reclamation site has marine deposits of average undrained shear strength 5.0 kPa and thickness of 10.0 m with a water depth of 10.0 m.
2. Unit weight of reclamation fill is 19.0 kN/m^3.
3. The factor of safety for the leading edge should not be less than 1.2.

Task

Determine the minimum length of the leading edge if the first layer of fill to be placed is 3.0 m thick.

Solution

Equation 3.3 may be applied for preliminary analysis. The length of the leading edge of the reclamation is taken as the length from the toe to the crest of the embankment L_e in the equation.

The minimum length of leading edge can therefore be calculated using Equation 3.3 as below:

Undrained shear strength, c_u	=	5.0 kPa
Submerged unit weight, $\gamma_{(submerged)}$	=	19.0-10.1 = 8.9 kN/m^3
		(unit weight of seawater is 10.1 kN/m^3)
Mud thickness, D	=	10.0 m
Fill thickness, H	=	3.0 m
Required FOS, F	=	1.2

Assume the friction angle of the fill is 30^o, then

$$K_a = \frac{1-\sin 30^\circ}{1+\sin 30^\circ}$$

$$= 0.3$$

$$\alpha = -\frac{0.3(8.9)(3^2)(1.2)}{2(5.0)(L_e)} = \frac{-2.8836}{L_e}$$

Therefore,

$$\frac{(1.2)(8.9)(3.0)}{5.0} = 4 + \frac{(1 + \frac{-2.8836}{L_e})L_e}{10.0}$$

$$L_e = 27.0\text{ m}$$

For detailed analysis of stability, reference should be made to Section 3.1.3.

D.2 Example 2 – Estimation of Residual Settlement with Surcharge Preloading

Given

1. The loading variations of a small reclamation project, which involve filling to final formation level, application of surcharge preloading, removal of surcharge preloading and application of future imposed load, are shown in Figure D1.

2. Vertical drains are installed before fill placement and the programme of reclamation is assumed as follows:

Completion of vertical drain installation:	commencement of filling
Completion of filling to final formation level:	the end of the 9th month
Filling of surcharge preloading:	in the 10th month
Removal of surcharge preloading:	end of the 22nd month
(removal up to final formation level +4.5 mPD)	
Application of future imposed load:	in the 24th month

Note : The time in the above programme is measured from commencement of filling.

3. Design data of the reclamation is shown below:

Table D2.1

Reclamation Profile	
Seabed level	-8.0 mPD
Mean sea level	+1.3 mPD
Final formation level	+4.5 mPD
Surcharge preloading thickness	5.0 m
Marine deposits data	
Thickness, H	10.0 m
Drainage condition	Double drainage
Unit weight, γ_{MD}	16.0 kN/m^3
Coefficient of consolidation (vertical), c_v	1.5 m^2/yr
Coefficient of consolidation (horizontal), c_h	1.5 m^2/yr
Compression ratio, CR	0.29
Recompression ratio, RR	0.06
Coefficient of secondary consolidation, $C_{\alpha\varepsilon}$	0.005
Fill data	
Unit weight, γ_{fill}	19.0 kN/m^2
Logarithmic creep compression rate, α	1.0%

Vertical drains data	
Type	Prefabricated band drains (100 mm wide x 5 mm thick)
Spacing	1.5 m
Pattern	Triangular and full depth
Future imposed load	
Future imposed load	20.0 kPa

<u>Task</u>

Carry out preliminary assessment of the following:

1. Final primary consolidation settlement due to reclamation fill and future imposed load.
2. Primary consolidation settlement occurred at end of 22nd month.
3. Secondary consolidation of marine deposit.
4. Creep of reclamation fill.
5. Residual settlement at end of 22nd month.

<u>Approach</u>

Preloading by surcharge load with band drain installation is applied to accelerate the consolidation settlement. The marine deposit is subject to time-dependent loading as shown in Figure D1. The loading due to reclamation fill is taken to be increased linearly and the non-uniform loading pattern is schematised as uniform loading pattern.

The marine deposit is divided into 10 sub-layers in the calculation and the degree of consolidation is calculated for each sub-layer. The primary consolidation settlement is obtained by summing up the settlement of each sub-layer.

Band drains are installed before commencement of filling and their function is assumed normal throughout the reclamation period and the surcharging period. The compression ratio, CR, is assumed to be constant for the whole marine deposit stratum and also remain constant in the process of primary consolidation. If the soil is variable, the depth should be represented by soil layers with different CR with reference to site investigation results.

N.B.

a. Only settlements in marine deposit and reclamation fill are considered and settlement in other soil stratum is neglected. In real case, settlement of all soil strata, including the alluvium clay layer, should also be considered.

b. The designer shall decide the most appropriate schematization for the loading sequence.
c. Alluvium sand is assumed underlying the marine deposit. As such, double drainage condition can be assumed.

Settlement estimation

(1) Final primary consolidation settlement due to reclamation fill and future imposed load

The final formation level of the reclamation is +4.5 mPD after removal of surcharge preloading. Taking into account the settlement, which is initially assumed to be 3.0 m (it should be adjusted based on subsequent calculated settlement), the total effective stress increment due to the loading of reclamation fill is given by:

Depth of fill below MSL = +1.3 mPD - (-8.0 mPD) + 3.0 m = 12.3 m
Depth of fill above MSL = +4.5 mPD - 1.3 mPD = 3.2 m

$$\Delta\sigma_{v(\text{fill})} = 12.3(19.0 - 10.1) + 3.2(19.0) = 170.3 \text{ kN/m}^2$$

$$\Delta\sigma_{v(imposed\ \ load)} = 20.0 \text{ kN/m}^2$$

The primary consolidation settlement due to the loading of reclamation fill and future imposed load of each sub-layer (without surcharge preloading) is calculated as follows:

Table D2.2

Sub-layer	z (mPD)	Initial effective stress (kN/m^2)	Total effective stress increment (kN/m^2)	Settlement (m)
1	-8.5	2.95	190.3	0.53
2	-9.5	8.85	190.3	0.39
3	-10.5	14.75	190.3	0.33
4	-11.5	20.65	190.3	0.29
5	-12.5	26.55	190.3	0.26
6	-13.5	32.45	190.3	0.24
7	-14.5	38.35	190.3	0.22
8	-15.5	44.25	190.3	0.21
9	-16.5	50.15	190.3	0.20
10	-17.5	56.05	190.3	0.19
			Total	**2.87**

The final primary consolidation settlement due to the loading of reclamation fill and future imposed load, $S_{p(final)}$, is estimated to be 2.87 m. This amount is close to the initial assumed value of 3 m and therefore recalculation is not made in this example.

(2) Primary consolidation settlement occurred at end of 22nd month

(2a) Degree of consolidation due to the loading of reclamation fill (with vertical drains) at the end of the 9.5th month (see Figure D1)

Consolidation due to vertical drainage alone

Applying the correction for construction time (see Section 4.3.3) and taking into account the construction time for filling the surcharge mound,

$$t = 10 - \frac{9}{2} - \frac{1}{2} = 5\,\text{months} = 0.42\ \text{years}$$

$$T_v = \frac{c_v t}{d^2} = \frac{1.5 \times 0.42}{5^2} = 0.025$$

(for double drainage, d = 10.0/2 = 5.0 m)

Equation B2a is applied to calculate the degree of consolidation for each sub-layer of the marine deposit as below:

Table D2.3

m	M	$\frac{2}{M}\sin\left(\frac{Mz}{H_{dr}}\right)\exp(-M^2T_v)$ with z (m) =									
		0.5	1.5	2.5	3.5	4.5	5.5	6.5	7.5	8.5	9.5
0	0.5π	0.187	0.543	0.846	1.067	1.182	1.182	1.067	0.846	0.543	0.187
1	1.5π	0.111	0.241	0.172	-0.038	-0.217	-0.217	-0.038	0.172	0.241	0.111
2	2.5π	0.039	0.039	-0.039	-0.039	0.039	0.039	-0.039	-0.039	0.039	0.039
3	3.5π	0.008	-0.001	-0.006	0.009	-0.004	-0.004	0.009	-0.006	-0.001	0.008
	sum	0.344	0.821	0.974	0.999	1.000	1.000	0.999	0.974	0.821	0.344
	ui	0.656	0.179	0.026	0.001	0.000	0.000	0.001	0.026	0.179	0.656

Consolidation due to horizontal drainage alone

$$t = 0.42\ \text{year}$$

$$\text{D} = 1.05 \times 1.5 = 1.58\,\text{m}$$

$$T_h = \frac{c_h t}{D^2} = \frac{1.5 \times 0.42}{1.58^2} = 0.25$$

$$d' = \frac{2(5+100)}{\pi} = 66.8 \text{ mm}$$

$$n = \frac{D}{d'} = 23.7$$

$$F(n) = \frac{n^2}{n^2-1}\ln n - \frac{3n^2-1}{4n^2}$$
$$= 2.42$$

From Equation B3,

$$U_h = 1 - \exp[\frac{-8T_h}{F(n)}]$$
$$= 1 - \exp[\frac{-8\times 0.25}{2.42}]$$
$$= 0.56$$

Consolidation under combined vertical and horizontal drainage

From Equation B4, $U_f = U_h + U_v - U_h U_v$, the degree of consolidation at each sub-layer is calculated as below:

Table D2.4

Layer	z (mPD)	ui
1	-8.5	0.849
2	-9.5	0.639
3	-10.5	0.571
4	-11.5	0.561
5	-12.5	0.560
6	-13.5	0.560
7	-14.5	0.561
8	-15.5	0.571
9	-16.5	0.639
10	-17.5	0.849

(2b) Effective stress increment at the end of the 9.5th month (see Figure D1)

The marine deposit is subject to different degree of consolidation at different depth. The calculation of the primary consolidation settlement for the first sub-layer is illustrated as an example as follows.

The initial effective stress at the center of the first marine deposit sub-layer is given by

$$\sigma_{v0,z=0.5m}' = 0.5(16.0 - 10.1) = 2.95\ \mathrm{kN/m^2}$$

The first estimate of settlement occurs at end of 9.5th month is taken to be 1.9 m (it should be adjusted based on subsequent calculated settlement). Taking into account the effect of settlement in the change of effective stress increment (larger portion of reclamation fill is submerged below seawater level), the effective stress increment due to the loading of reclamation fill at the centre of the first sub-layer is given by:

Depth of fill below MSL = +1.3 mPD - (-8.0 mPD) + 1.9 m = 11.2 m
Depth of fill above MSL = +4.5 mPD - 1.3 mPD - 1.9 m = 1.3 m

$$\begin{aligned}\Delta\sigma_{v(\mathrm{fill})} &= [11.2\times(19.0-10.1)+1.3\times19.0]\times0.849 \qquad \text{(See Table D2.4)}\\ &= 124.4\times0.849\\ &= 105.6\ \mathrm{kN/m^2}\end{aligned}$$

From Equation 4.1a, the primary consolidation settlement in the first sub-layer (H = 1.0 m for each sub-layer) due to the loading of reclamation fill is given by:

$$\begin{aligned}S_{p(\mathrm{fill}),\ \mathrm{z=0.5m,\ 9.5th\ month}} &= HCR\log\left(\frac{\sigma_{v0}'+\Delta\sigma_{v(\mathrm{fill}),\ \mathrm{z=0.5m,\ 9.5th\ month}}}{\sigma_{v0}'}\right)\\ &= 1.0(0.29)\log\left(\frac{2.95+105.6}{2.95}\right)\\ &= 0.45\ \mathrm{m}\end{aligned}$$

Check the magnitude of the primary consolidation settlement of the soil:

Table D2.5

Layer	z (mPD)	Initial effective stress (kN/m^2)	Effective stress increment (kN/m^2)	Settlement at the end of 9.5th month (m)
1	-8.5	2.95	105.6	0.45
2	-9.5	8.85	79.5	0.29
3	-10.5	14.75	71.1	0.22
4	-11.5	20.65	69.7	0.19
5	-12.5	26.55	69.7	0.16
6	-13.5	32.45	69.7	0.14
7	-14.5	38.35	69.7	0.13
8	-15.5	44.25	71.1	0.12
9	-16.5	50.15	79.5	0.12
10	-17.5	56.05	105.6	0.13
			Total	1.96

The primary consolidation settlement due to the loading of reclamation fill at the end of 9.5th month is estimated to be 1.96 m and is close to the initial estimate of 1.9 m. Therefore, recalculation of the effect of settlement on the change in effective stress increment is not carried out in this example.

(2c) The degree of primary consolidation due to reclamation fill and surcharge preloading (with vertical drains) at the time of surcharge removal (end of 22nd month)

Consolidation due to vertical drainage alone

Applying the correction for construction time (see Section 4.3.3),

$$t = 13 - \frac{1}{2} = 12.5\,\text{months} = 1.04\,\text{years}$$

$$T_v = \frac{c_v t}{d^2} = \frac{1.5 \times 1.04}{5.0^2} = 0.062$$

For rough estimation, Equation B2a is assumed valid to calculate the degree of consolidation for each sub-layer of the marine deposit. They are calculated as below:

<u>Table D2.6</u>

m	M	$\frac{2}{M}\sin\left(\frac{Mz}{H_{dr}}\right)\exp(-M^2T_v)$ with z (m) =									
		0.5	1.5	2.5	3.5	4.5	5.5	6.5	7.5	8.5	9.5
0	0.5π	0.171	0.496	0.773	0.974	1.079	1.079	0.974	0.773	0.496	0.171
1	1.5π	0.049	0.106	0.076	-0.017	-0.095	-0.095	-0.017	0.076	0.106	0.049
2	2.5π	0.004	0.004	-0.004	-0.004	0.004	0.004	-0.004	-0.004	0.004	0.004
3	3.5π	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	sum	0.224	0.606	0.844	0.953	0.988	0.988	0.953	0.844	0.606	0.224
	ui	0.776	0.394	0.156	0.047	0.012	0.012	0.047	0.156	0.394	0.776

Note :

Strictly speaking, Equation B2a is not valid as the excess pore water pressure is not constant throughout the marine deposit layer. The excess pore water pressure, u_e, at depth z after time t is given by:

$$u_e = \sum_{n=1}^{n=\infty}\left(\frac{1}{d}\int_0^{2d} u_i \sin\frac{n\pi z}{2d}dz\right)\left(\sin\frac{n\pi z}{2d}\right)\exp\left(-\frac{n^2\pi^2 c_v t}{4d^2}\right)$$

Alternatively, the method developed by M. Carter for Macau International Airport can be adopted to determine the time curve for the degree of consolidation. For details, see pp 35-40 in Transactions, Volume 3, Number 1, the Hong Kong Institution of Engineers.

Consolidation due to horizontal drainage alone

$$t = 1.05\text{ year}$$

$$\text{D} = 1.05\times1.5 = 1.58\text{ m}$$

$$T_h = \frac{c_h t}{D^2} = \frac{1.5\times1.05}{1.58^2} = 0.63$$

$$\text{d'} = \frac{2(5+100)}{\pi} = 66.8\text{ mm}$$

$$n = \frac{D}{d'} = 23.7$$

$$F(n) = \frac{n^2}{n^2-1}\ln n - \frac{3n^2-1}{4n^2}$$
$$= 2.42$$

From Equation B3,

$$U_h = 1 - \exp[\frac{-8T_h}{F(n)}]$$
$$= 1 - \exp[\frac{-8 \times 0.63}{2.42}]$$
$$= 0.88$$

Consolidation under combined vertical and horizontal drainage

From Equation B4, $U_f = U_h + U_v - U_h U_v$, the degree of consolidation at each sub-layer is calculated as below:

Table D2.7

Layer	z (mPD)	ui
1	-8.5	0.973
2	-9.5	0.927
3	-10.5	0.899
4	-11.5	0.886
5	-12.5	0.881
6	-13.5	0.881
7	-14.5	0.886
8	-15.5	0.899
9	-16.5	0.927
10	-17.5	0.973

(2d) Primary consolidation settlement due to reclamation fill and surcharge preloading (with vertical drain) at the time just immediately after surcharge removal (end of 22nd month)

The first estimate of settlement occurs at end of 22nd month is taken to be 3.0 m (it should be adjusted based on subsequent calculated settlement). The effective stress increment due to the loading of reclamation fill at the centre of the first sub-layer is given by:

Depth of fill below MSL = +1.3 mPD - (-8.0 mPD) + 3.0 m = 12.3 m
Depth of fill above MSL = 5.0 m + 4.5 mPD - 1.3 mPD – 3.0 m = 5.2 m

The effective stress increment if all pore pressure is dissipated at end of 22nd month:

$$\Delta\sigma_v = 12.3(19.0 - 10.1) + 5.2(19.0) = 208.3\,\text{kN/m}^2$$

Effective stress increment at end of 9.5 months = 105.6 kN/m^2 (see Table D2.5)

As only 97.3% pore water pressure is dissipated at end of 22nd month (see Table D2.7), therefore

$$\Delta\sigma_{v,\,\text{z=0.5m, 22th month}} = 105.6 + (208.3 - 105.6)\times 0.973 = 205.5\ \text{kN/m}^2$$

From Equation 4.1a, the primary consolidation settlement in the first layer (H = 1.0 m for each sub-layer) due to the loading of reclamation fill and surcharge preloading (with vertical drains) is given by:

$$\begin{aligned} S_{p,\,\text{z=0.5m, 22th month}} &= HCR\log\left(\frac{\sigma_{v0}' + \Delta\sigma_{v,\,\text{z=0.5m, 22nd month}}}{\sigma_{v0}'}\right) \\ &= 1.0(0.29)\log\left(\frac{2.95 + 205.5}{2.95}\right) \\ &= 0.54\ \text{m} \end{aligned}$$

The settlement for the primary consolidation settlement due to reclamation fill and surcharge preloading (with vertical drains) at the time of surcharge removal (end of the 22nd month) for all the sub-layers are calculated as follows:

Table D2.8

Sub-layer	z (mPD)	Initial effective stress (kN/m^2)	Effective stress increment (kN/m^2)	Settlement at the end of 22nd month (m)
1	-8.5	2.95	205.5	0.54
2	-9.5	8.85	198.9	0.40
3	-10.5	14.75	194.4	0.33
4	-11.5	20.65	192.5	0.29
5	-12.5	26.55	191.9	0.27
6	-13.5	32.45	191.9	0.24
7	-14.5	38.35	192.5	0.23
8	-15.5	44.25	194.4	0.21
9	-16.5	50.15	198.9	0.20
10	-17.5	56.05	205.5	0.19
			Total	**2.90**

The total consolidation settlement due to reclamation fill and surcharge preloading (with vertical drains) at the end of the 22nd month is estimated to be 2.9 m.

Note : The settlement at the end of the 22nd month is equal to 2.9 m which is very close to the initial estimate of 3.0 m. Further recalculation is therefore not required in this example.

(2e) Residual settlement due to primary consolidation

As the effective stress increment in the marine deposit is greater than that due to the future loading, the marine deposit is considered over-consolidated. A rebound will occur after removal of surcharge loading. The change in settlement will then follow recompression curve when the future imposed load (20.0 kN/m^2) is applied on the reclamation.

Effective stress due to self weight of marine mud (at centre of soil layers)
= 5.0 x (16.0 - 10.1) = 29.5 kN/m^2

Depth of fill below MSL = +1.3 mPD - (-8.0 mPD) + 2.9 m = 12.2 m
Depth of fill above MSL = +4.5 mPD - 1.3 mPD = 3.2 m
(Surcharge removed to formation level +4.5 mPD only)

Effective stress due to fill
= 12.2 x (19.0 - 10.1) + 3.2 x 19.0 = 169.4 kN/m^2

Effective stress due to imposed load = 20.0 kN/m^2

The order of the maximum recompression, when the rebound is fully developed after a certain period of time, can be approximated by:

$$
\begin{aligned}
S_{\max,recompression} &= H.RR.\log(\frac{\sigma_{v0}'+\Delta\sigma_v}{\sigma_{v0}'}) \\
&= 10.0(0.06)\log(\frac{29.5+169.4+20}{29.5+169.4}) \\
&= 25 \text{ mm}
\end{aligned}
$$

The residual settlement due to primary consolidation is therefore equal to about 25 mm.

N.B. The future imposed load will be applied shortly (2 months later) after the removal of the surcharge loading. As a result, the rebound will not be fully developed. The order of recompression should be smaller than 25 mm in principle.

(3) Settlement due to secondary consolidation in marine deposit

The residual settlement due to secondary consolidation is:

$$S_{s,t(\text{residual})} = C_{\alpha\varepsilon} H.\log(\frac{t_2 - t_o}{t_1 - t_o})$$

$$= 0.005 \times 10.0 \times \log(\frac{50 - 9.5/12}{22/12 - 9.5/12})$$

$$= 84\ \text{mm}$$

(4) Settlement within reclamation fill

Due to the primary settlement, the thickness of the reclamation fill is taken to be 15.5m. The residual settlement of reclamation fill due to creep at end of 22nd month:

$$S_{c,t(\text{residual})} = H\alpha \log(\frac{50 - \frac{t_c}{2}}{t - \frac{t_c}{2}})$$

$$= 15.5 \times 0.01 \times \log(\frac{50 - 9/2/12}{22/12 - 9/2/12})$$

$$= 237\ \text{mm}$$

(5) Residual settlement at end of 22nd month

Total residual settlement at the time of removal of surcharge is estimated to be

= $S_{s,p(\text{residual})} + S_{s,t(\text{residual})} + S_{c,t(\text{residual})}$

= 25 + 84 + 237

~ $\underline{\underline{350\ \text{mm}}}$

D.3 Example 3 – Monitoring of Pore Pressure for Stability during Fill Placement

Given

A piezometer installed in the marine deposit layer revealed abnormalities during the fill placement works. Increase of excess pore water pressure was found larger than the predicted value as shown in Figure D2. Pause of filling works was considered necessary (see Section 5.3.2).

The dimensions of vertical drains were the same as those in Example D.2. The coefficient of consolidation (horizontal) was taken as 1.5m^2/year.

Task

Estimate the time required (t) for the excess pore water pressure to fall back to the normal magnitude.

Approach

(1) Define the abnormal excess pore pressure as the initial excess pore pressure (u_o).
(2) Define the target normal excess pore pressure (u_e).
(3) Determine the required degree of consolidation (U_h) for dissipation of excess pore pressure from u_o to u_e.
(4) Back calculate the time factor (T_h) from the required U_h.
(5) Obtain the actual time t required from T_h.

Solution

Initial excess pore pressure, u_0	=	20.0 kN/m^2	(see Figure D2)
Excess pore water pressure at time t, u_e	=	14.9 kN/m^2	(see Figure D2)
Pore pressure to be dissipated, d_p	=	20.0 – 14.9 = 5.1 kN/m^2	

Neglecting vertical drainage, average degree of consolidation required to dissipate the above pore pressure is given by:

$$U_h = \frac{5.1}{20} = 0.255$$

The value of *F(n)* in Example D.2 was used because the dimensions and layout of vertical drains were the same. Therefore, *F(n)* = 2.42.

From Equation B3,

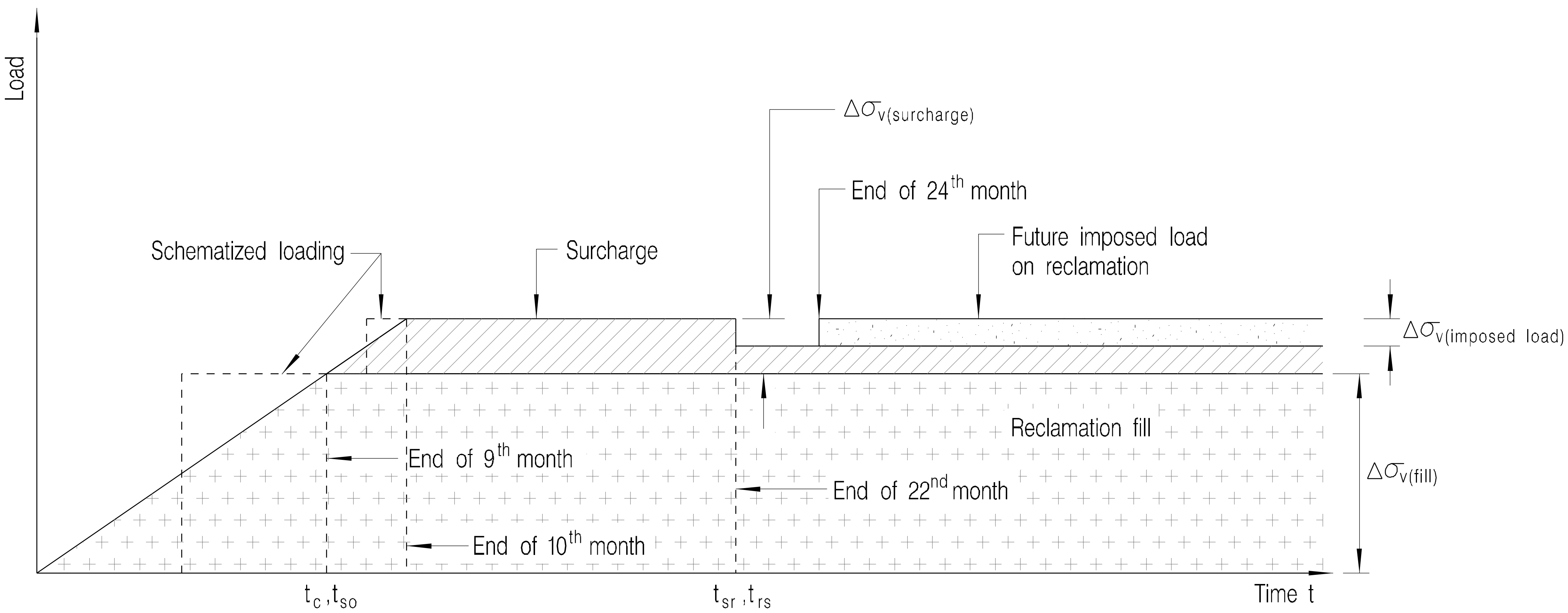
$$
\begin{aligned}
T_h &= -\frac{F(n)}{8}\ln(1-U_h) \\
&= -\frac{2.42}{8}\ln(1-0.255) \\
&= 0.09
\end{aligned}
$$

Therefore,

$$t = \frac{D^2 T_h}{c_h} = \frac{2.1(0.09)}{1.5} = 0.13 \text{ year}$$

i.e. Time required for the excess pore pressure to fall back to the normal magnitude is about $1\frac{2}{3}$ months.

LIST OF FIGURES



t_c = Time for completing fill placement or the construction period = end of the 9th month

t_{so} = Time for starting application of surcharge load = start of the 10th month

t_{sr} = Time for completing surcharge removal = end of the 22nd month

t_{rs} = The point at which residual settlement is required = end of the 22nd month

Figure D1 – Example 2: Loading Variations of the Reclamation

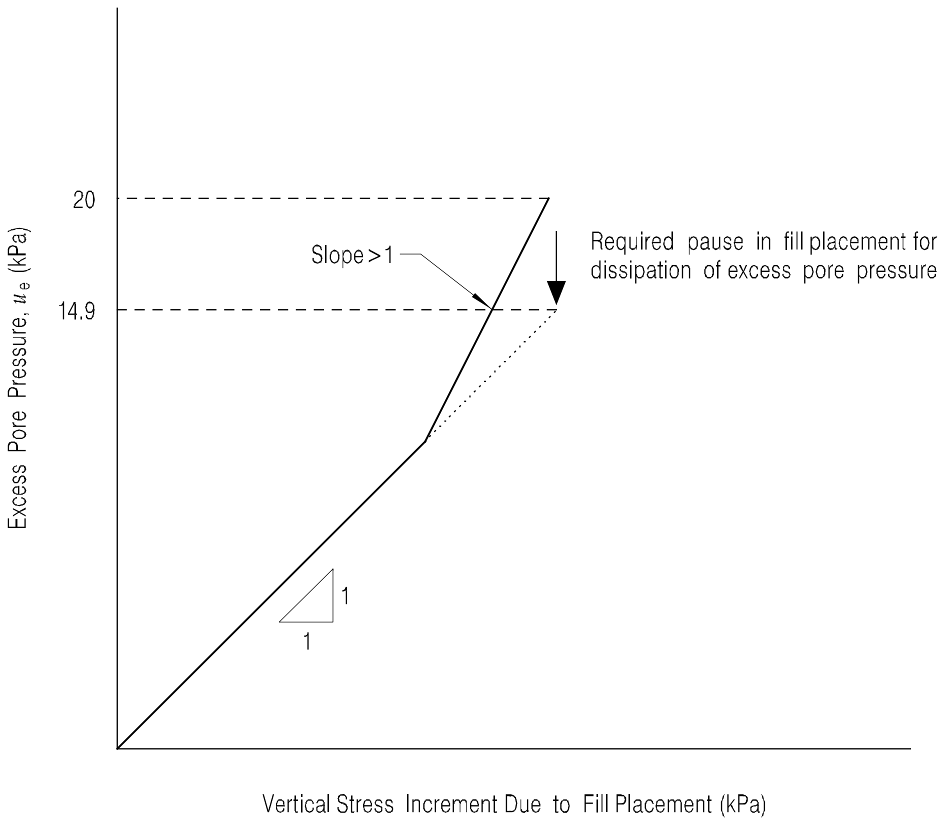


Figure D2 – Example 3: Pore Pressure Monitoring Data

GLOSSARY OF TERMS AND SYMBOLS

GLOSSARY OF TERMS

Adsorbed water. A soil particle carries an electrical charge, which attracts ions to neutralize its net charge. Such attracted ions, which are weakly held on the particle surface and can be readily replaced by other ions, are termed exchangeable ions. The water attached to the exchangeable ions attracted to the soil surface is called adsorbed water and is not free to flow under gravity, unlike the pore water.

Blanket layer. A layer of free draining granular material placed on top of the seabed in a drained reclamation. It is used primarily for spreading the load of fill placed on top of it and to provide a filter layer to prevent coarser material being punched into the soft marine deposits. If vertical drains are used, it also provides a drainage layer that ensures vertical drainage from the underlying marine deposits.

Compression ratio. Slope of the virgin compression line in a strain-log effective stress curve.

Creep. Long-term time dependent compression of soil under constant loading.

Differential settlement. Settlement of one location relative to another, usually expressed as a ratio (e.g. 1:300) of the distance between the two locations.

Displacement method. An old reclamation method involving direct tipping of fill from trucks onto the seabed without dredging away or treating the underlying soft deposits.

Drained reclamation. Reclamation with the soft deposits left in place with or without surcharge preloading and/or vertical drains.

Dredged reclamation. Reclamation with the soft deposits removed before the fill is placed.

Dynamic compaction. A fill densification method which involves repeated dropping of heavy weights from a height onto the ground surface.

Effective preconsolidation pressure. The maximum effective vertical stress that has acted on a soil layer in the past.

Excess pore water pressure. The increase in pore water pressure above the final or steadystate value.

Mud wave. Excessive displacement of mud in a reclamation following successive slip failures caused by loading too quickly, loading in an unbalanced manner, or inadequate drainage of the mud.

One-dimensional (1-D) strain. A type of consolidation in which deformation and water flow are all in one direction and the deformation in orthogonal directions is considered to be zero. It is generally applicable to reclamation works where the thickness is small compared to the lateral extent.

Primary consolidation. A process of gradual reduction in the volume of a fully-saturated soil of low permeability due to the drainage of pore water.

Recompression. The compression of over-consolidated soil when the effective stress is smaller than the effective preconsolidation pressure.

Recompression ratio. Average slope of the recompression line in a strain-log effective stress curve.

Residual settlement. The amount of remaining settlement due to primary consolidation and secondary consolidation (or creep) of the sub-soils and fill that would occur from a given time onwards, after the completion of filling works.

Secondary consolidation. Long-term settlement of clay that occurs under constant effective stress (thought to be due to squeezing out of the adsorbed water and rearrangement and/or deformation of clay particles).

Smear effect. Effect of reduced permeability of the soil surrounding a vertical drain due to the disturbance caused by the installation of the vertical drain.

Surcharge preloading. A ground treatment method, which can both accelerate the consolidation of sub-soil layers and densify the fill, by placing surcharge (usually additional fill) temporarily on top of the reclamation.

Vertical drains. Vertical drainage conduits (such as band drains) or wells (such as sand drains) installed within the soils at spacing closer than the drainage distance for vertical flow, which shorten the drainage path and hence accelerate the consolidation process.

Vibrocompaction. A fill densification method for granular soils, which involves penetration and controlled retraction of a vibrating tool in the soil.

Virgin compression. The compression of a normally consolidated soil due to an increase in effective stress.

Well resistance. The finite permeability of vertical drains with respect to the surrounding soil.

GLOSSARY OF SYMBOLS

α	Logarithmic creep compression rate (%)
β_0 , β_1	Parameters obtained from Asaoka's Graphical Method for prediction of primary consolidation settlement
γ	Unit weight
γ_d	Dry unit weight
$\gamma_{d\ max}$	Dry unit weight of soil in densest condition
$\gamma_{d\ min}$	Dry unit weight of soil in loosest condition
δ_H	Lateral deformation
$\Delta\sigma_v$	Applied vertical load
$\Delta\sigma_{v(\mathrm{fill})}$	Applied loading due to the reclamation fill
$\Delta\sigma_{v(\mathrm{imposed\ load})}$	Applied loading due to the future imposed load
$\Delta\sigma_{v(\mathrm{surcharge})}$	Applied loading due to the surcharge mound
μ	Correction factor for $(c_u)_{FV}$
σ_p'	Effective preconsolidation pressure
σ_{v0}'	Initial effective stress
τ_f	Shear strength along the selected surface in slip surface analysis
$C_{\alpha\varepsilon}$	Coefficient of secondary consolidation in terms of strain
C_c	Compression index
c_h	Coefficient of consolidation (horizontal)
CR	Compression ratio
C_r	Recompression index
c_u	Undrained shear strength
c_v	Coefficient of consolidation (vertical)
$(c_u)_{FV}$	Measured undrained shear strength from field vane
D	Diameter of the equivalent cylinder of soil drained by a vertical drain

d	Length of longest drainage path
d'	Drain diameter of a vertical drain
D_r	Relative density
e	Void ratio
e_0	Initial void ratio
e_{max}	Void ratio of the soil in loosest condition
e_{min}	Void ratio of the soil in densest condition
F	Factor of safety
H	Layer or fill thickness
I_p	Plasticity index
K_a	Coefficient of active pressure
l	Length of vertical slices along the selected surface in slip surface analysis
L_e	Length from the toe to the crest of embankment
n	spacing ratio of a vertical drains system
r	Moment arm for τ_f in slip surface analysis
RR	Recompression ratio
s	Settlement
S_c	Settlement of fill due to creep
$S_{c,t(residual)}$	Residual settlement of fill due to creep at time t
S_p	Ultimate primary consolidation settlement
$S_{p,t}$	Primary consolidation settlement achieved at time t
$S_{p,t(residual)}$	Residual settlement of a sub-soil layer due to primary consolidation at time t
$S_{residual,t}$	Residual settlement of a reclamation due to the sub-soil and fill at time t
S_s	Settlement due to secondary consolidation

$S_{s,t(residual)}$	Residual settlement of a soil layer due to secondary consolidation after a given time *t*
t	Time from the instantaneous application of a total stress increment
t_0	Start of the time for secondary-consolidation calculations
t_1	Time at which secondary consolidation begins
t_c	Time for completing fill placement or the construction period
T_h	Time factor (horizontal drainage)
T_v	Time factor (vertical drainage)
u	Excess pore water pressure
U_f	Effective degree of consolidation due to vertical and horizontal drainage
U_h	Degree of consolidation (horizontal drainage)
U_t	Degree of consolidation at time *t*
U_v	Degree of consolidation (vertical drainage)
W	Weight of vertical slices in slip surface analysis
x	Moment arm for *W* in slip surface analysis