Republic of the Union of Myanmar
Ministry of Construction, PW

The Project for Improvement of Road Technology in Disaster Affected Area in Myanmar

Soft Ground Treatment Manual

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Japan International Cooperation Agency
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The Republic of the Union of Myanmar
Department of Highways, Ministry of Construction

Project for Improvement of Road Technology
in Disaster-affected Areas in Myanmar

Soft Ground Treatment Manual

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This manual is modified from the original for Myanmar.

The original written in Japanese is

“Japan Road Association (2012); Soft Ground Treatment Manual”.

All of the figures and the tables in this manual are from the original unless otherwise specified.

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Introduction

1-1 Damages on Soft Ground

Earthwork structures on soft ground that has insufficient shear strength and bearing capacity can suffer various damages such as deformation of the structures and the surrounding ground, or unevenness of road surfaces due to not only slip failure of an embankment and consolidation settlement but also earthquakes as indicated in Table 1-1.

Table 1-1 Problems of soft ground

Stage	Problems	Impact on Road Surface and Embankment	Impact on Surrounding Ground	Impact on Facilities / Structures
Construction Stage	Settlement	- Increase in embankment volume - Residual settlement - Deformation of embankment	- Swelling - Settlement - Move outward - Move inward	- Damage to structures - Uneven settlement
	Slip Failure	- Slip Failure - Level difference Cracks	- Wide-ranging settlement	- Damage to surrounding Structures - Displacement of abutment
Maintenance Stage	Residual Settlement (Long-term Settlement)	- Road surface iirregularities - Poor trafficability - Poor road surface drainage	- Settlement in surroundings - Poor drainage in surroundings - Settlement of structures	- Level difference at approach to structures - Poor trafficability - Impact on structures - Settlement of culvert - Poor drainage - Shortage of capacity - Displacement of retaining wall - Permeation of surface water - Disintegration of backfill soil and pavement - Deformation of retaining wall

When an earthwork structure constructed on soft ground, the structure is sometimes suffered major damage such as slip failure or settlement. Typical damage to embankments on soft ground are shown in Figure 1-1. Figure 1-1 d shows a case of ground settlement near an abutment or culvert associated with any of a. to c., where road traffic can be disturbed more seriously than an ordinary embankment section due to the displacement, cracking, settlement, and gaps. The followings are potential problems that require particular attention on road earthworks in the area of soft ground:

a. Slip failure

b. Insufficient bearing capacity

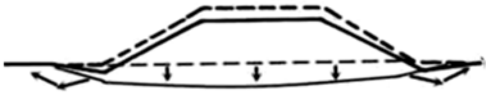
c. Settlement and deformation of the ground and the surroundings

d. Damage to culverts or retaining walls

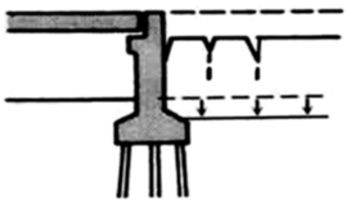
e. Slip failure and liquefaction due to earthquakes

a. Slip Failure (Rotational Failure)

b. Insufficient Bearing Capacity (Base Failure)



c. Settlement



d. Settlement of Embankment behind Abutment

Figure 1-1 Examples of damages to embankment on soft ground

(1) Slip Failure

The dead weight of the earthwork structure, vibration from the construction machinery, or traffic loads could induce slip failure and damage to not only the earthwork structures but also the surrounding ground and facilities.

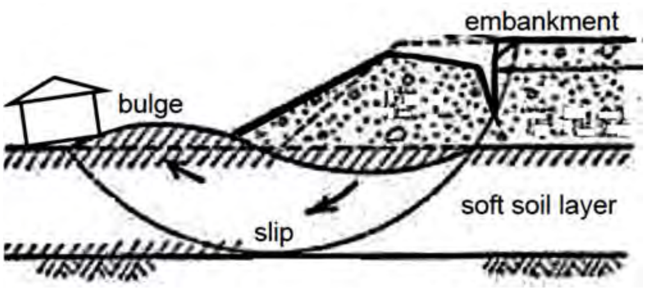


Figure 1-2 Slip failure affect on neighbours (TCP)

The safety of earthwork structures on the soft ground is generally evaluated by stability analysis of slip failure. However, the result of the stability analysis of slip failure depends on the accuracy of the soil investigation result, the soil properties obtained and the method of the stability analysis. Figure 1-3 shows an example of a slip failure caused by a road embankment affecting surrounding structures in Japan.

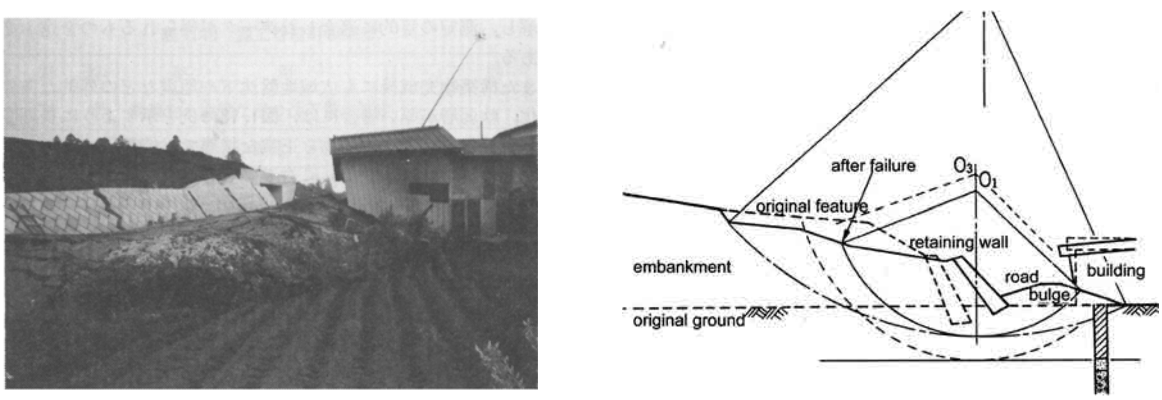


Figure 1-3 Example of slip failure involving the surrounding ground *(Japan)* [1)]

Figure 1-4 shows an example of a slip failure caused by an embankment beside a river affecting surrounding the river in Japan. The ground deformation around the embankment affected river bed and retaining walls during work period. Figures 1-5 and 1-6 are examples of ground deformations caused by road embankment works. The impact of ground deformation for surrounding buildings or important facilities nearby during work period and after completion should be studied.

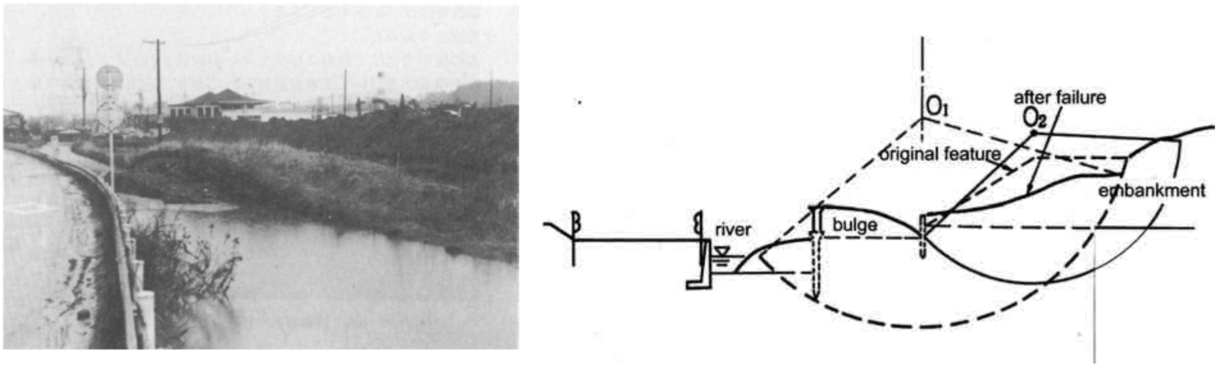


Figure 1-4 Example of slip failure involving river *(Japan)* [1)]

Figure 1-5 Clacks and depression caused by slip failure *(Myanmar)*

Figure 1-6 Swelling caused by slip failure [(TCP)]

(2) Settlement and Deformation of Soft Ground

The load of earthwork structure on soft ground can cause ground settlement or horizontal movement which have worse influence on not only abutments, retaining walls, or culverts negatively but also surrounding facilities (Figure 1-7). Therefore, in the series of construction of an earthwork structure, it is necessary to adopt appropriate measures to prevent damage in the vicinity, to design taking into account the settlement of the surrounding ground, or to construct the structures after sufficient settlement.

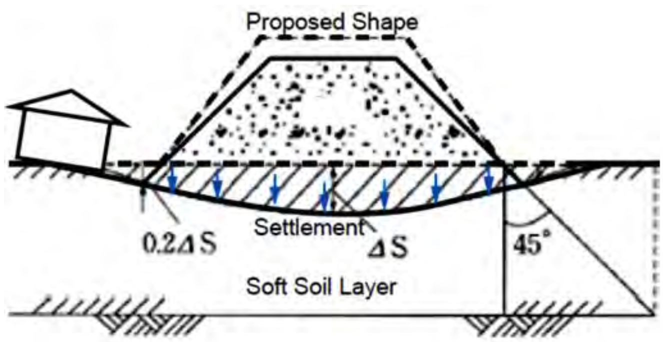


Figure 1-7 Embankment settlement (TCP)

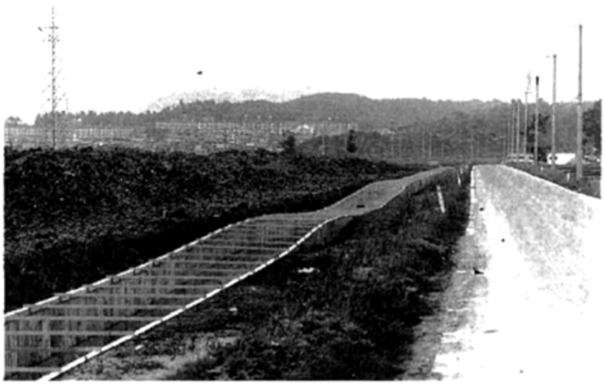


Figure 1-8 Deformation of a water channel affected by ground settlement

Continuous excessive ground settlement after the completion of the pavement on the embankment affects the road surface flatness, pavement damage and trafficability, also induces drainage malfunctions. Particularly, uneven settlement at a joint between a bridge/culvert and an embankment causes disturbance on the road trafficability. The settlement after the completion of the construction works is caused by consolidation settlement of the soft ground induced by the embankment loads. Estimation of the settlement in the design stage is therefore based on the calculation of the consolidation of the soft ground layer. In determining the allowable residual settlement the following various relevant factors should be considered.

- type of road
- type of pavement
- location of settlement
- influence on trafficability
- influence on transverse and longitudinal gradients of drainages
- influence on the structures and facilities nearby on the surrounding ground
- difficulty of repair work
- cost performance

Figure 1-9 Example of settlement on road surface

Figure 1-10 Example of settlement on road surface [4)]

Construction of an embankment on the soft ground where serious settlement is anticipated for a long period or in wide area is mostly difficult to control the residual settlement within the target ranges or is extremely uneconomical. In that case, it is necessary to consider some residual settlement in the maintenance stage by taking proactive measures, such as installing approach slabs, carrying out patching repair and constructing temporary pavement.

A low embankment of less than 2.0 to 2.5 m in height can be damaged to the pavement and the road flatness due to uneven settlement by impacts of the traffic load. This is because influence of the traffic load is relatively large comparing with the embankment weight, and because the low embankment can be affected by the groundwater, which lowers the shear strength of the low embankment.

In the road section in cut and bank, serious uneven settlement on the road surface can be induced even after completion of construction of the road because of great changes of thickness of soft material along the longitudinal or transverse direction, or of existence of sandy gravel layer in a shallow part in locally. Necessary measures should be taken appropriately in such sections based on the site conditions through the topographical or geological survey, since it is difficult to identify such locations by investigation borings before construction

(3) Damages to Culverts and Retaining Walls

Culverts or retaining walls constructed at the same time as an embankment can cause serious deformation during or after construction. Figure 1-11 is typical damages to retaining walls on soft ground.

Figure 1-12 is an example of water stagnation as a result of settlement of a crossing culvert with settlement of an embankment.

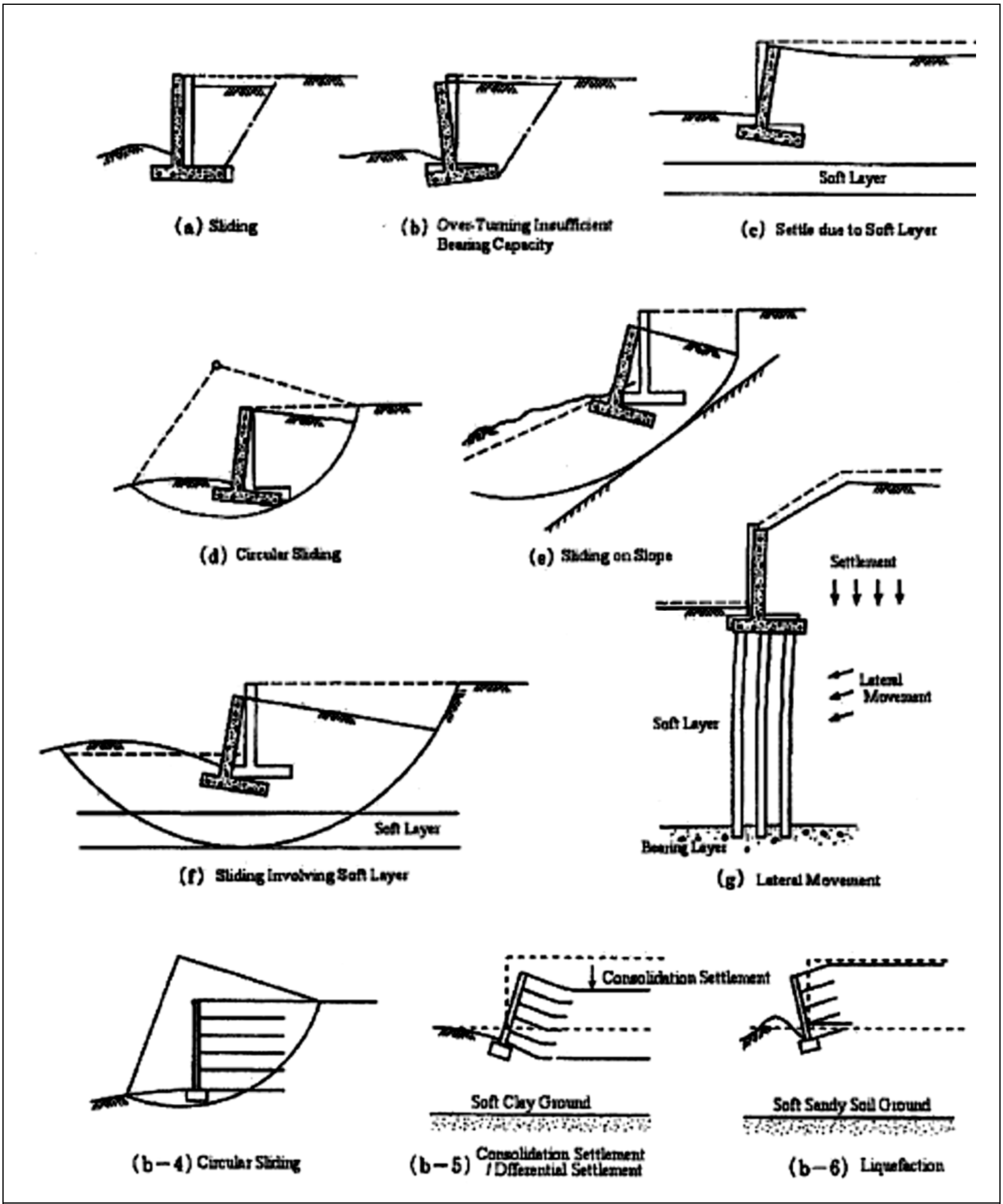


Figure 1-11 Various types of damage to retaining wall on soft ground [2)]

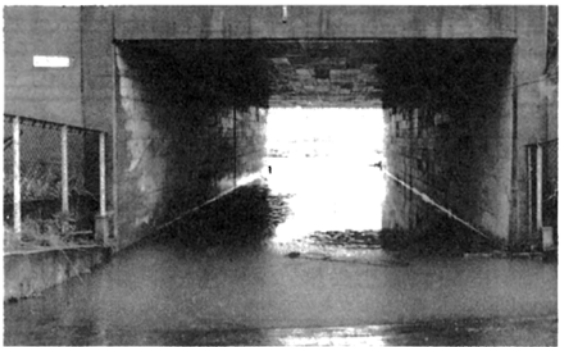


Figure 1-12 Water stagnation at a road crossing point due to culvert settlement

A pile foundation could be considered to prevent displacement or settlement of a culvert or retaining wall up to the bearing stratum. However, if the soft ground is very thick, the pile should be very long, and pile foundation construction becomes uneconomical. In addition, the foundation will receive negative friction due to embankment settlement or non-uniform embankment earth pressure. A crossing culvert with a pile foundation will not sink, and this will cause a gap with the settled road surface. Therefore it is not desirable to apply pile foundations for culverts. It is recommended to construct road embankments in advance and to excavate the embankment in order to construct culverts after sufficient settlement. Even if this kind of method is used, some uneven settlement may still be induced on the road surface if the cover pavement is thin, but no major problems will arise if the culvert cover is thick.

When a ditch type road is constructed in soft ground with a high groundwater level, the cost could increase for the soft ground treatment and the extra costs for the drainage of groundwater after the road opening. If it is necessary to construct such a ditch type road for unavoidable reasons, it is necessary to review the following items and to take appropriate measures depending on the road importance level:

- Uneven settlement of the foundation of the ditch type road;
- Uplift by groundwater or seepage of water;
- Drainage of groundwater or rainwater; and
- Stability in an earthquake.

(4) Damages Due to Earthquake

When an earthwork structure constructed on soft ground experiences a large earthquake, the stability may be reduced and the structure may suffer major damage such as slip failure or settlement. Major types of damage to embankments on soft ground caused by a large earthquake are almost same as ordinal damage to embankment as shown in Figure 1-1. Sufficient attention should be paid in case a road is built on soft ground since the soft ground can get the following problems by an earthquake.

a. The strength of soft ground composed of cohesive soil or peat can be increased as consolidation progresses over time. As a result, the ground may be stable against earthquakes. Therefore, the soft ground receives seismic motion after a certain period from the completion of an embankment; road functions will not be affected seriously. It is necessary to pay careful attention to liquefaction on fill material or sand layers deeper than the groundwater level, as shown in Figure 1-13, where the groundwater level is high and settlement by consolidation is serious, or where a depression was filled in.

b. Even if the foundation ground is composed of cohesive soil, high sensitivity ratio clay or silt may suffer a great degradation of strength due to the effect of seismic motion.

c. Saturated, loose sand can be liquefied due to the decrease in the shear strength of the ground with the effect of seismic motion. The thicker the saturated sand layer, the more serious the effect of liquefaction can be on the road functions.

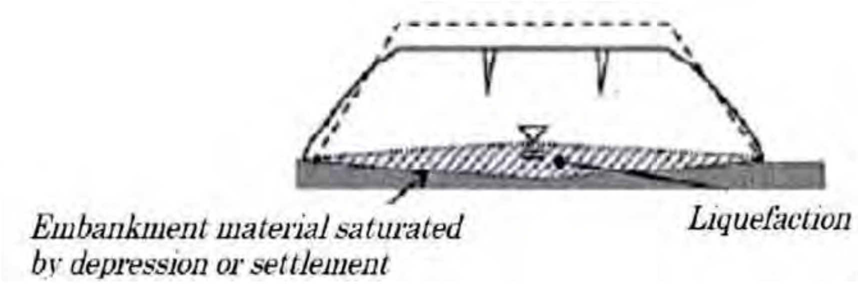


Figure 1-13 Damage of an embankment on soft ground due to an earthquake

d. In most cases, the liquefaction is induced in the ground composed of alluvial sandy soil. However, liquefaction is induced even in low-plasticity silty sand or gravel soil with an average grain size of over 2 mm, as seen in the Hyogoken Nanbu Earthquake in Japan in 1995 and recent earthquakes. Particularly, attention should be paid to the fact that liquefaction can cause great deformation on a coastline or slope foundations.

Figure 1-14 Road surface deformation due to liquefaction [5)]

Figure 1-15 Road collapse due to earthquake [6)]

1-2 Soft Ground

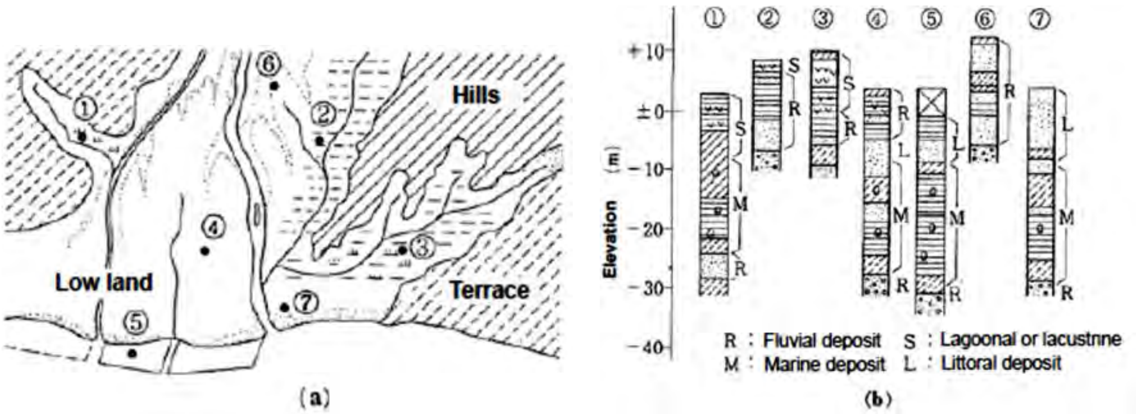
(1) Definition of Soft Ground

Soft ground is generally composed of fine particle soil such as clay and silt, porous highly organic soil, or loose sand. These highly problematic soft ground layers show smaller strength and higher compressibility if the layer is younger, the groundwater level is higher, and overburden pressure from upper layers is smaller. Therefore, stratification or soil properties of soft ground are variable depending on the conditions of sedimentation, which are shown as geomorphic features.

The ground composed of cohesive soil with N-value of 4 or smaller can cause settlement or unstable conditions, and the ground composed of sandy soil with N-value of below 10–15 can suffer liquefaction. The cohesive soil with N-value of 4 or smaller and the sandy soil with N-value of below 10-15 is called the soft ground. The earth structure constructed on the soft soil can be unstable due to the insufficient bearing capacity and shear strength, or the large compressibility. The required strength or settlement characteristics of the soft ground are different in every case, since the load on the ground and the allowable displacement are variable depending on the type or scale of the earthwork structure.

(2) Distribution of Soft Ground

The soft ground areas are generally classified topographically into a. aggraded buried valleys, b. back marshes, c. narrow valleys between hills, d. delta lowlands, e. reclaimed land, f. coastal sand bars, and g. natural levees. Figures 1-16 and 1-17 show type of soft ground and typical location of soft ground, and Table 1-2 shows characteristics of the ground, and characteristics of the soils.



1 Aggraded buried valley, 2 Back marsh, 3 Narrow valley between hills, 4 Delta lowland, 5 Reclaimed land, 6 Natural levee, 7 Coastal sandbar

Figure 1-16 Distribution of soft ground and stratification

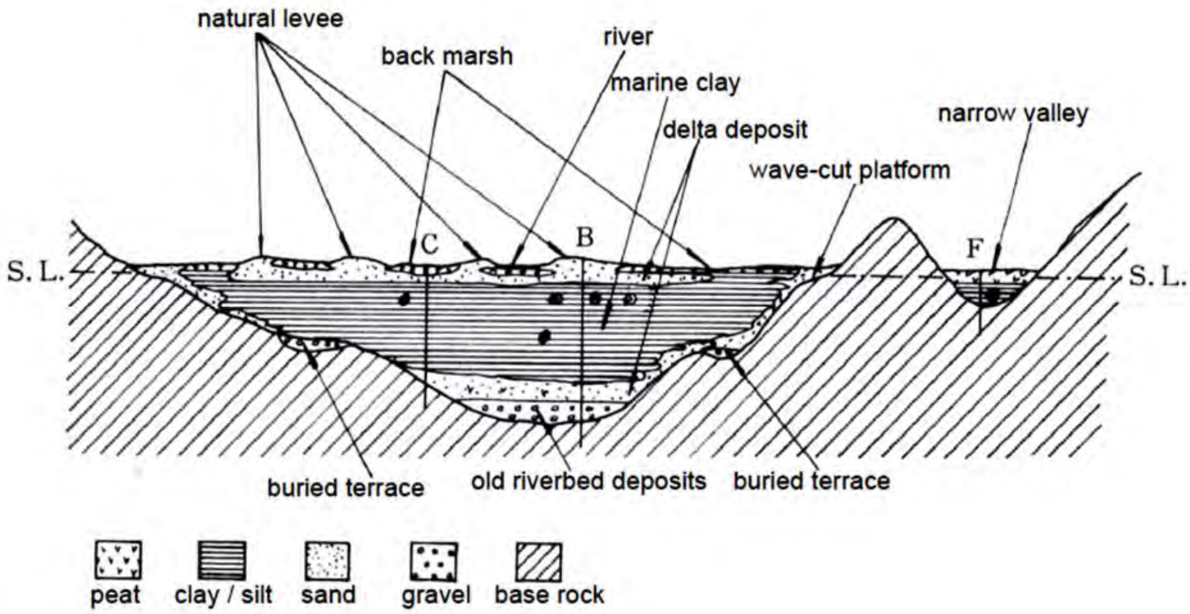


Figure 1-17 Typical cross sections of soft ground area [3)]

Table 1-2 Distribution and characteristics of soft ground

Distribution	Characteristics of soft ground
Aggraded buried valley	This terrain was originally a drowned valley (a valley topography sunken under the sea by sea-level rise) buried with deposits and then exposed on the ground surface again by upheaval. It is often formed as a thick soft soil layer with a mixture of a large volume of organic matter.
Back marsh	Back marsh is wet ground behind a natural levee. It is often characterized by alternation of cohesive and sandy gravel layers. The area is sometimes overlaid by thick layers of fluvial organic soil and cohesive soil.
Narrow valley between hills	A colluvial valley, buried valley, small drowned valley (drowned valley with its bay mouth closed by a coastal sandbar), branch valley, etc. It is often overlain by thick layers of lagoonal or lacustrine peat or organic soil and underlain by thick layers of marine clay. Thickness of the deposit material in this type of valley rarely exceeds 10 m.
Delta lowland	Lowland ground formed at a river delta of a gently flowing river. It is often composed of alternating cohesive soil and sand. It can form large-scale soft ground underlain by a thick layer of marine cohesive soil.
Reclaimed land	Reclaimed land is artificial land reclaimed from the sea or wet land. Problems often arise where the land was reclaimed on soft sea bottom with thick disturbed cohesive soil and silt, and where compaction is not yet sufficient. The land reclaimed with sandy soil has high potential of liquefaction.
Coastal sandbar Natural levee	Coastal sandbar is the ground along coastal line, and the natural levee is the ground along a major river. They are good ground mostly, but sometimes they are overlain by thick layers of lose sand and underlain by thick clayey layers.
Other	River channels, primary path of a river, embankments in wet land, rims of natural levees, lowlands between sand dunes, and boundaries between sand dunes and lowlands are often characterized by loose sand deposit, and the liquefaction potential is high.

(3) Classification of Soft Soil

Table 1-3 shows the normal ranges of geotechnical properties of soft ground types obtained in past road earthworks in Japan, such as natural water content

w_n, natural void ratio e_n, unconfined compression strength q_u, and N-values, which are classified into three types of ground: peat ground, cohesive ground, and sandy ground.

Table 1-3 Classification of soft soil and soil properties

Ground Classification	Soil Classification			Soil Property				Typical Location
				W_n (%)	e_n	q_u (kN/m^2)	N-value	
Peat	High organic soil {Pm}	Peat (Pt)	Fibrous high organic soil	≥ 300	≥ 7.5	≤ 40	≤ 1	Buried valley Back marsh Narrow valley
		Muck (Mk)	Highly decomposed high organic soil	300 - 200	7.5 – 5			
Cohesive soil	Fine-grained soil {Fm}	Organic soil (O)	Under A-line of Plasticity Chart	200 - 100	5 – 2.5	≤ 100	≤ 4	Buried valley Back marsh Narrow valley Delta low land Reclaimed land
		Volcanic cohesive soil {V}	Under A-line of Plasticity Chart					
		Silt {M}	Under A-line of Plasticity Chart; large dilatancy	100 - 50	2.5 – 1.25			
		Clay {C}	Under or near A-line of Plasticity Chart; small dilatancy					
Sandy soil	Coarse-grained soil {Cm}	Sand with fine-grained soil {SF}	15 – 50% of fine-grained soil (≤ 75 µm)	50 - 30	1.25 – 0.8	-	≤ 10	Delta low land Reclaimed land Natural levee Coastal sandbar
		Sand {S}	< 15% of fine-grained soil (≤75 µm)	≤ 30	≤ 0.8	-		

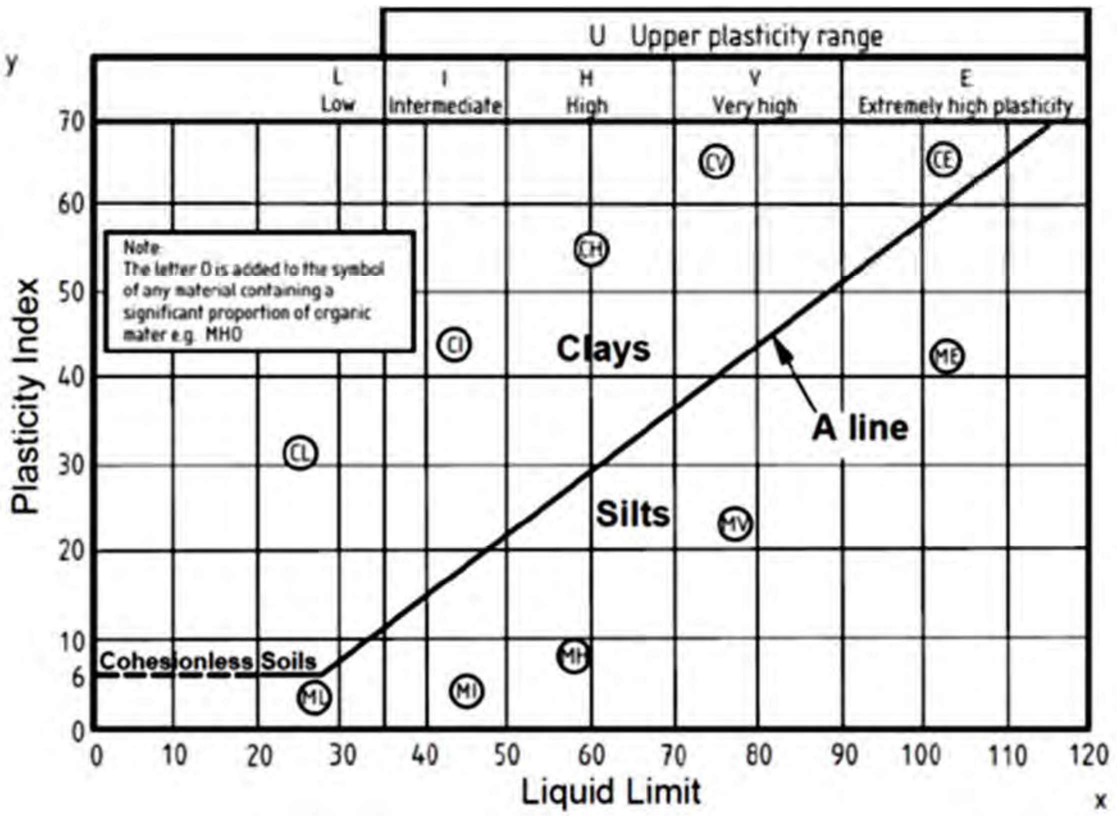
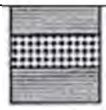


Figure 1-18 Plasticity chart [7)]

When the ground consists of such a soil shown in Table 1-3, necessary measures shall be taken in the investigation, design, and construction stages to solve the anticipated problems, since the ground may not have sufficient bearing capacity as the foundation of earthwork structures. The soil composition of typical soft ground and its characteristics are shown in Table 1-4.

Table 1-4 Geological structure of typical soft ground and its characteristics

Type	Diagram	Characteristics	Typical Location
Clay layers		Most typical type of soft ground; it is composed of only clay or clay with organic content. Its q_u value increases in proportion to the depth from the ground surface. Marine clay presents in lower part of thick layer has high sensitivity. And once this marine clay is disturbed by improvement works, it takes long time to recover the original strength, and for consolidation settlement.	Buried valley, Reclaimed land
Sand layer at Upper parts		Sand layer of about 3–5 m in thickness lies on top of the ground has less stability-related problems, since the drainage from the layer is good. Liquefaction may sometimes be a problem.	Coastal sandbar, Natural levee
Sand layer at Inter-bedded		Terrestrial clay and marine clay is deposited in the upper part and lower part respectively, and an intermediate sand layer lies in-between. Good drainage from the layer causes less stability-related problems. After embankment on this ground, however, long-term settlement often continues in the lower clay layer.	Back marsh, Delta lowland
Peat layer at upper part		Peat layer covers on top part of the subsurface. Terrestrial layer often lies below the peat layer till 10–15 m deep, and it changes to marine clay in deeper. Clay of immediately under the peat layer often has stability problems since its high sensitivity. If there is no clay layer under the peat layer, consolidation settlement complete rapidly, even though the settlement amount is large.	Buried valley, Back marsh
Peat layer at Inter-bedded		Clay layer is on the surface and peat layer lies under it. Because of its composition of complex strata, the depth distribution of the q_u value is unclear. If an embankment is built on it, the strength and behaviour of organic clay under the peak play important roles.	Back marsh

(4) Soft Ground Distribution in Myanmar

In the north, the Hengduan Mountains form the border with China. Hkakabo Razi, located in Kachin State, at an elevation of 5,881 meters (19,295 ft), is the highest point in Burma. Many mountain ranges, such as the Rakhine Yoma, the Bago Yoma, the Shan Hills and the Tenasserim Hills exist within Burma, all of which run north-to-south from the Himalayas. The mountain chains divide Burma's three river systems, which are the Irrawaddy, Salween (Thanlwin), and the Sittaung rivers. The Irrawaddy River, Burma's longest river, nearly 2,170 kilometers (1,348 mi) long, flows into the Gulf of Martaban. Fertile plains exist in the valleys between the mountain chains. The majority of Burma's population lives in the Irrawaddy valley, which is situated between the Rakhine Yoma and the Shan Plateau. The soft soil in Myanmar is dominant in lowland shown as dark green in Figure 1-19 topographic map of Myanmar. Lowland areas are alluvium material deposit area in delta, along shoreline, along major rivers such as Irrawaddy River. These areas are non-rock area as shown in Figure 1-20. Non-rock area consists of mostly alluvial clay and sand.

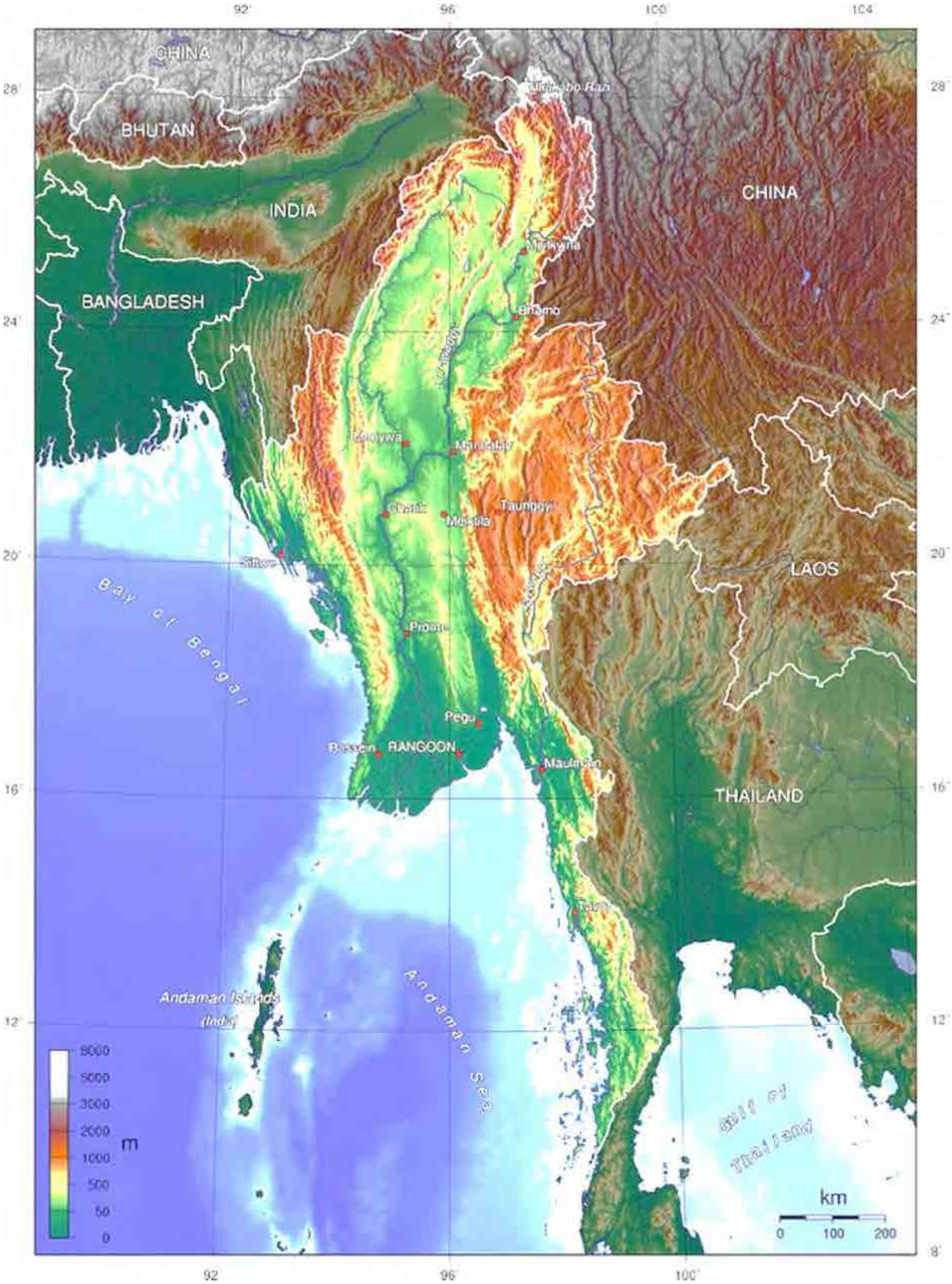


Figure 1-19 Topographic map of Myanmar [8)]

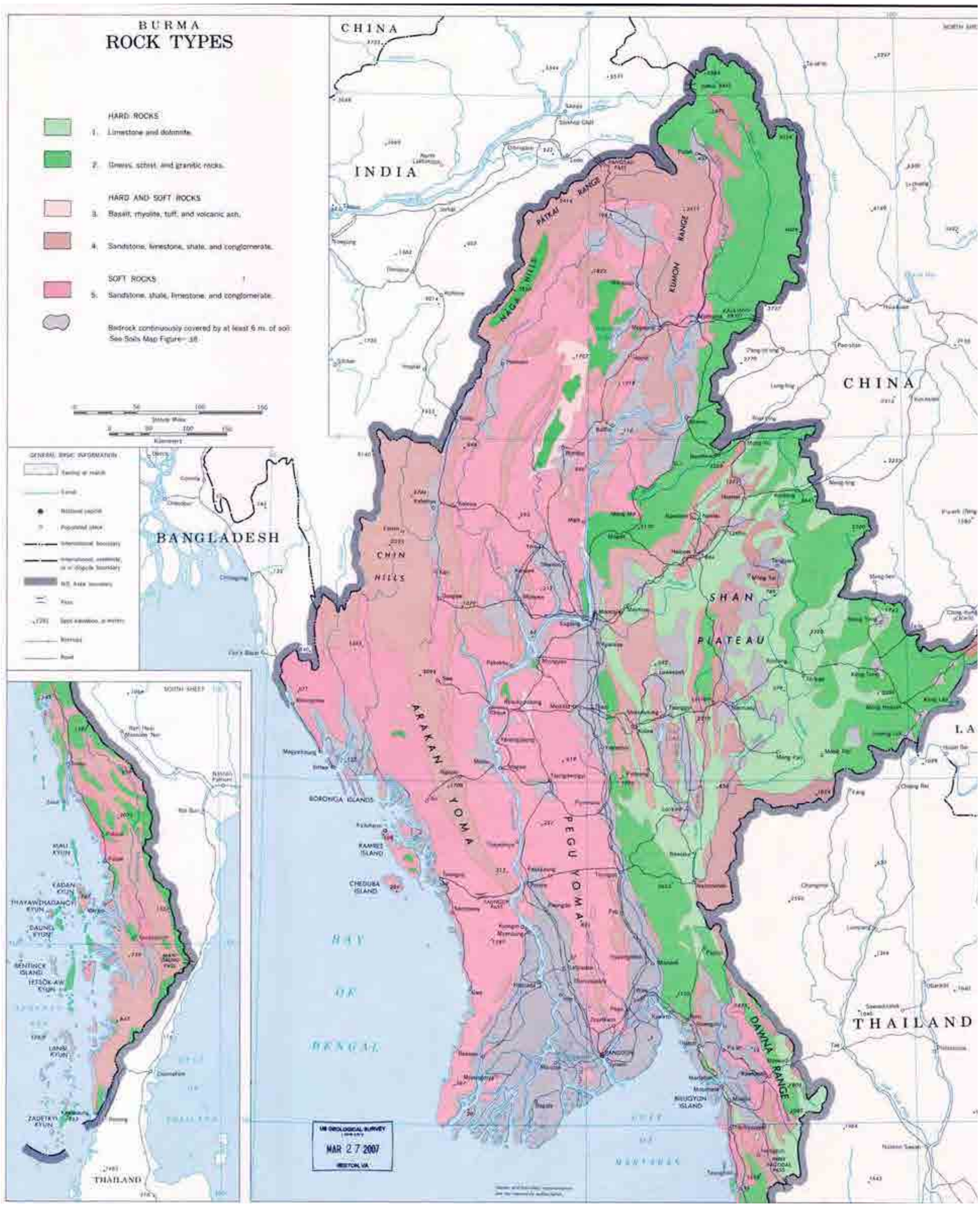


Figure 1-20 Geology of Myanmar [9)]

1-3 Concepts of Soft Ground Treatment

1-3-1 Purpose and Concept

Soft ground treatment shall be conducted; (a) to construct earthwork structures safely and economically on soft ground where construction would be difficult if no improvement measures were applied, and where the maintenance of performance could be impossible even if earthwork structures were constructed; and (b) to ensure necessary functions to maintain the safety and smoothness of road traffic during road service period. Earthwork structures constructed on soft ground will affect not only the structure but also the surrounding ground. Controlling settlement and deformation of the surrounding ground is therefore required in addition to maintaining the functions of the earthwork structure.

1-3-2 Procedure of Soft Ground Treatment

The entire process of soft ground treatment is shown in Table 1-5. Investigation and analysis of soft ground and improvement work shall be implemented systematically while enhancing the precision based on the analysis of each stage of the road earthworks. The procedure for road earthworks on soft ground begins from investigation of the stratigraphic condition of the ground and the physical properties of each layer, and the possibility of constructing an earthwork structure should be studied during road planning, whether without any measures, or with some necessary measures. Soft ground treatment shall be studied if safety during the construction period is not satisfied or the structure's stability or intended function or performance during road service period cannot be ensured due to ground settlement. In such a case, the purposes and expected effects of the work, such as stabilization or settlement control, shall be clarified, and several specific improvement methods shall be selected to meet these purposes and effects. This process may include a change of the earthwork structure or revision of the entire schedule depending on the situation. With respect to the selected measures, the most suitable method for the designed earthwork structure and the ground shall be studied by executing additional investigations for applicability, cost comparison, and implementation effect; and through the analysis of construction conditions such as procurement of materials and machinery or restrictions of the space necessary for construction.

Since soft ground treatment has un-ignorable impacts on road construction both in terms of expenditure and time, it is important to develop a road construction plan with soft ground measures from the beginning.

Table 1-5 Work procedure of soft ground treatment

	Work Progress	stage
1	Preliminary investigation	Investigation / Study
2	Preliminary study (confirmation of existence of soft ground)	
3	Basic investigation	
4	Basic study *Rough study on soil strength, stability and settlement* *Decision of necessity of detailed investigation*	
5	Detailed investigation	
6	Detailed Design *General matter for detailed study* *Study on soil strength, stability, settlement and deformations* *Decision of necessity of soil treatment works* *Decision of type of soil treatment work and design*	Design of Earthwork Structure Design of Soft Ground Treatment
7	Preparation of work plan and control levels	
8	Preparation of instrumentation and monitoring	Execution
9	Work control by monitoring (work control, monitoring on stability / settlement /deformations)	
10	Decision of control level in maintenance stage	
11	Inspection , repair / restoration	Maintenance

(1) Investigation and Planning Stage (Initial and Preliminary Design)

a. The longitudinal and transverse alignment of a road are determined by economic and social factors in most cases. It is desirable to avoid a soft ground area from the viewpoints of road construction and maintenance. In the event that a soft ground area cannot be avoided, it must be given appropriate consideration, such as setting the height of an embankment as low as possible under the given conditions, or eliminating large-scale ground improvement to prevent the deformation in the surrounding ground or structures.

b. The selection of an improvement method in the planning stage has an extremely large impact on the cost or period of soft ground treatment compared with the design stage. Since the construction type of a structure or the type of improvement work is mostly decided before starting the design stage, there is little room for new changes, modifications, or improvements in that stage. Therefore, even in the planning stage it is important to study the possibility of various methods including the introduction of new technologies and methods based on the performance-based framework introduced in this Manual as described in "Chapter 3: Study for Design of Earthwork Structures on Soft Ground".

c. When the study of soft ground is generally conducted in the entire process of road planning together with that of other structures such as bridges, study from various aspects is required on the selection and design of improvement work from economical and rational points of view, and it takes a large amount of time. Therefore, it is necessary to start the study of soft ground treatment from the earliest stage in the road planning process.

(2) Detailed Design Stage

a. The properties of soft ground are generally complicated. When a study of soft ground is made based on the analysis of the soil investigation results, it is extremely difficult to have a sure understanding of the properties of the soft ground and to correctly predict any phenomenon that could arise with respect to the earthwork structure during the process of construction or during road service period. Therefore, on designing soft ground treatment, it is desirable to carry out trial construction as required, to estimate the design parameters by back analysis based on the trial construction results, and then to carry out the design of the improvement method or earthwork structures.

b. On determining the improvement method, it is preferable to study construction methods with the maximum utilization of the characteristics of the ground, such as an extra-fill method, preloading method, or slow loading method, to increase the ground strength by the consolidation of the ground, and, if the construction period is restricted, to consider other methods.

c. The framework of performance-based design has been introduced in this Manual. This revised Manual provides basic principles on the use of any analysis methods, design methods, materials, or structures to satisfy the requirements for earthwork structures that are not specified in the conventional requirements. In future, creative ideas applicable to soft ground treatment are expected to enhance accurate and economical design and construction by the adoption of design verification based on recognized, reasonable analysis methods or by the implementation of trial construction or observation procedures to ensure the precision of the ground parameters and to improve the ground information in the initial design stage.

d. There are various kinds of methods in soft ground improvement, such as conventional methods and newly developed methods that have been disseminated recently. The adoption of new methods or technologies may realize more efficient and economical construction of earthwork structures with appropriate quality. On introducing a new method or technology, comprehensive study of the total economic performance is necessary, not only for reducing the period and cost of the construction stage, but also for material quality, durability and easiness of maintenance, and for avoiding traffic congestion due to traffic control in the maintenance stage. Furthermore, it is recommendable to conduct appropriate performance evaluation of a method or technology proposed for adoption through trial construction in order to compare it with the conventional technology if necessary. Information on new technologies may be obtained from NETIS (New Technology Information System).

(3) Construction Stage

a. In constructing an earthwork structure on soft ground, the construction plan shall be meticulously and carefully developed on studying the properties of the soft ground in order to ensure sufficient quality of the soft ground treatment and the earthwork structure. It is also necessary to conduct the monitoring on the ground behavior through the entire work period in order to control the works. If any issues are found after completion of the soft ground treatment, it is very difficult to remedy the work. Therefore, it is important to conduct reliable quality and construction control during the construction period in order to ensure the performance of the earthwork structure. In this case, it is necessary to pay attention to the following points.

- Develop a construction plan based on the conditions and requirements of the design drawings and specifications, the construction period, the quality, and the progress of the works.
- Settlement and stability should be monitored based on work control levels using the instruments installed at appropriate locations.

If a large value exceeding the control level has been detected from the observation results, there is a possibility of ground deformation increasing rapidly up to destruction. The construction plan shall be re-examined immediately, and emergency measures shall be taken as required, and then the construction method shall be corrected as necessary.

b. Since work conducted on soft ground causes various kinds of problems during the work for the environment of the people in the vicinity, including noise and vibration, and unexpected damage to facilities along the road, it is necessary to take reliable and urgent preventive measures according to their importance on considering the conditions of the surrounding ground and structures.

(4) Maintenance Stage

Earthwork structures constructed on soft ground will suffer settlement and deformation continuously even during road service period, and this will affect road functions unless appropriate maintenance is executed. Therefore, it is important to identify and designate locations where continuous settlement and deformation are predicted as the high priority inspection points during the process of maintenance planning, and to utilize reference data about soil investigation or field observation which was prepared in the design and construction stages.

1) Watanabe, et al. (1986) ; Soft Ground Treatment Works Pocket Book (Japanese)

2) Public Works Research Institute Japan (2004) ; Manual for Highway Earthworks in Japan (English)

3) Industrial Research Center of Japan (1981); Soft Ground Handbook (Japanese)

4) (https://www.pwri.go.jp/jpn/webmag/wm025/seika.html)

5) (Imamura; http://www.ajiko.co.jp/yomimono/imamura05.html)

6) (Public Works Research Institute Japan; http://jiban.ceri.go.jp/naiyou1.html)

7) British Standard

8) (Wikipedia; http://en.wikipedia.org/wiki/Burma)

9) (http://www.mappery.com/map-of/Myanmar-Burma-Rock-Types-Map)

* TCP : Technical Cooperation Project (JICA Expert Team)

Chapter 2

Investigation and Study

2-1 Procedures of Investigation

2-1-1 Concepts of Investigation

(1) General Process of Investigation

Investigation of soft ground and study of treatment in the process of road construction are conducted considering related structures. Since the necessary types of information or the contents or precision of the analysis of road earthworks vary for each stage, the necessary investigations and studies are conducted sequentially to ensure appropriate design, construction, and maintenance. On conducting investigation and study of soft ground, various kinds of information are required, including geotechnical conditions, topography, geology, soil properties, nearby structures, surface water conditions, and meteorological conditions. Methods for investigations and studies related to geotechnical conditions are described in this Manual.

Preliminary and basic investigations are conducted in the planning stage and the basic design stage, respectively. Also, initial study and preliminary study should follow each investigation. Detailed design of earthwork structures and soft ground treatment can be developed based on detailed investigation and additional investigation. On performing detailed design, if any problems are found in the contents or precision of the information, an additional investigation and trial construction are conducted. Investigations are conducted even in the construction stage and maintenance stage. This Chapter describes the investigation procedures, from the planning stage to the maintenance stage, necessary for construction of an earthwork structure on soft ground, together with the contents and methods of these procedures. It also describes preliminary and basic study during the planning stage and basic design stage, including whether soft ground treatment is necessary or not and the selection of appropriate methods. The soil investigation in each stage is outlined as follows. Tables 2-1a, b, c show the relationships of investigations and studies for each stage.

1	Planning stage	Preliminary investigation Preliminary study
2	Basic design stage	Basic investigation Basic study
3	Detailed design stage	Detailed investigation Additional investigation Detailed design Trial construction
4	Construction stage	Investigation during construction stage
5	Maintenance stage	Investigation during maintenance stage

Table 2-1a Investigations and analysis in planning stage to decide route alignment

<table>
<tr><td rowspan="2">Preliminary Investigation</td><td>Method</td><td>• Collection and study of existing material
• Field reconnaissance</td></tr>
<tr><td>Outcome</td><td>• Schematic understanding of distribution of soft ground
• Engineering properties of soft ground</td></tr>
<tr><td rowspan="2">Preliminary Study</td><td>Method</td><td>• Estimation of critical embankment height
• Estimation of total settlement</td></tr>
<tr><td>Outcome</td><td>• Evaluation of problem locations, and study of avoidance method
• Study of structural type of earthwork structure, and study of improvement method
• Refining of candidate routes</td></tr>
</table>

Table 2-1b Investigations and analysis in basic design stage to determine right-of-way

<table>
<tr><td rowspan="2">Basic Investigation</td><td>Method</td><td>• Collection and study of existing materials,
• Field reconnaissance, simple sounding tests, sampling, and soil tests for soil classification
• Implementation of soil investigations such as boring, sounding, In situ tests, sampling, laboratory soil tests, geophysical exploration</td></tr>
<tr><td>Outcome</td><td>• Distribution and engineering properties of the soft ground along the proposed route
• Collection of information for selection of the improvement method and determination of the work volume
• Estimation of soil properties from existing materials
• Estimation of soil properties from soil investigations to refine the candidate structural types and dimensions of earthwork structures and improvement methods</td></tr>
<tr><td rowspan="2">Basic Study (without sufficient ground information)</td><td>Method</td><td>• Calculation of critical embankment height
• Calculation of settlement of soft ground at the center of the embankment (immediate settlement generally not considered)</td></tr>
<tr><td>Outcome</td><td>• Refining of structural type, dimensions, and improvement methods of earthwork structure</td></tr>
<tr><td rowspan="2">Basic Study (with sufficient ground information)</td><td>Method</td><td>• Determination of construction conditions and banking rate
• Settlement calculation: Total settlement (analysis of primary consolidation, and instant settlement as required)
• Settlement rate
• Residual settlement
• Stability calculation (strength increase by consolidation is generally not considered)
• Schematic judgment of liquefaction</td></tr>
<tr><td>Outcome</td><td>• Refining of the candidate structural types and dimensions of earthwork structures and improvement work methods</td></tr>
</table>

Table 2-1c Investigations and design in detailed design stage

<table>
<tr><td rowspan="2">Detailed and Additional Investigation</td><td>Method</td><td>• Detailed investigation includes soil investigation along the centerline of the road in the entire route; additional investigation includes exploration at locations that need attention
• Boring
• Sounding
• Geophysical exploration
• Sampling
• Soil tests
• Field tests</td></tr>
<tr><td>Outcome</td><td>• Collection of information for detailed design of earthwork structure
• Setting of soil properties</td></tr>
<tr><td rowspan="2">Trial Construction</td><td>Method</td><td>• Measurement of earthwork structure and ground behavior</td></tr>
<tr><td>Outcome</td><td>• Confirmation of design constants or effects of improvement work</td></tr>
<tr><td rowspan="2">Detailed Design</td><td>Method</td><td>• Settlement calculation
• Total settlement (immediate settlement considered)
• Settlement rate
• Residual settlement (secondary consolidation considered as required)
• Stability calculation (strength increase due to consolidation considered)
• Seismic design (judgment of liquefaction, residual deformation, stability analysis)</td></tr>
<tr><td>Outcome</td><td>• Setting of detailed earthwork structure and improvement method
• Verification of required performance such as safety factor for settlement or stability calculation
• Preparation of design and contract documents</td></tr>
</table>

(2) Important Points of Soil Investigation

When soil investigation of soft ground should be conducted according to the basic investigation policy as described above and the following points require attention.

a. Planning for appropriate methods and the scale of an investigation are selected and proposed depending on the purpose of the investigation, the extent of the soft ground, the road structure, etc. Investigation results are checked from time to time with the progress of the investigation, to confirm whether the purpose can be fulfilled or not based on the original investigation plan. Also, the investigation plan is modified if necessary.

b. Since the soil properties and soil composition of soft ground are complicated and different from location to location, it is necessary to analyze individual boring data comprehensively while considering the total ground condition. On selecting investigation sites, it is necessary to check the following items.
c. surrounding topography; the geological conditions and the history of the soft soil; land uses such as farmland, houses, or waterways, as well as the locations of springs.
d. extent and depth of the soft ground.
e. earthwork structures of the proposed road, for the height, width, and shape of embankments.
f. location, structure type, foundation type, and size of surrounding buildings.

g. Settlement or stabilization of soft ground can be presumed from the geological process, the topography, the thickness of the soft soil, and the soil composition. Information on similar ground in the area is useful for soil investigation, since past experiences show similar phenomena on similar types of ground.

h. A detailed investigation is executed at a place that represents typical cross-sections.

i. The existence of a permeable layer among clay layers is very important to estimate the consolidation settlement period. However, it is not easy to identify which layer works as a permeable layer.

j. Geological structure and soil properties of natural ground are not simple and not even. Stability and settlement of soft ground is studied based on soil investigation. With structures made of artificial materials, such as iron or concrete, it is not difficult to construct safe structures according to the design calculation. However, with natural soil or ground, the acquired samples are from a limited portion of the entire ground even if the data are detailed, because the ground is composed of complicated materials. Also, the sample itself is disturbed during the sampling for soil investigation and soil testing. These limitations of soil investigation data and the relation of each of the individual test data items should be recognized when they are used. In addition, re-investigation or re-testing is considered if the results seem to be unreliable. It is important to observe the deformation or settlement, comparing it with the predicted behavior during the construction, in order to confirm the reliability of the investigation data or the design. Moreover, the results are fed back to the design and work processes under the monitoring.

k. It is difficult to accurately predict ground deformation or failure in advance. Recent progress in research technology has been remarkable in the aspect of ground processes from deformation to failure and the process to steady status through long-term settlement. However, a practical research method has not yet been established to quantitatively predict the behavior of the ground under a consistent concept about the methods for investigation, soil testing, and design and analysis. It is therefore necessary to know the limitations as well as advantages and disadvantages of the methods from investigation to study, and appropriate judgments are required based on a recognition of the uncertainties in the design or study values.

(3) Soil Investigation at Noticeable Soft Ground

Detailed investigations are required in the following conditions which often causes problems in stability or settlement on the soft ground.

- Location at proposed cut and bank
- Considerable variation in the thickness of the soft layer in the longitudinal or transverse direction of the road
- Inclined surface of base layer
- Presence of structures nearby
- Embankment for the approach to an abutment or adjacent to a culvert
- Loose sandy soil where liquefaction is predicted
- Soft ground where long-term settlement is expected

Detailed investigations are also required particularly when settlement or slip failure is predicted to have a serious impact on the surrounding area, or where the traffic volume is too heavy to conduct appropriate traffic control after the service commencement, or there is no alternative route in the event of a major disaster. Meticulous study and design are necessary particularly for the ground conditions described below.

Soft soil layer on inclined surface of firm layer

In case an embankment is built on soft soil layer which lies on inclines foundation, the thicker side of soft soil layer settles more than the thinner side, and slip failure can often be induced due to lateral flow. Therefore, careful study is expected. Investigation by sounding is conducted for the whole area, and boring investigation is executed at typical points. In the case of a transition area from a flat area to a mountainous area, or a valley area with complicated topography, the geologic structure is generally complicated. It is therefore necessary to examine not only topographic conditions but also the spread of the soft ground or the condition of the foundation by a sounding survey or any other necessary means.

Marine clay

Among marine clays, of which Ariake Clay is a typical example, there are some kinds of soft soil ground for which a remarkable decrease in their strength can be caused due to disturbance by construction work. These soils have high water content, high plasticity, and loose particle bonding. On encountering such soft ground, it is necessary to use a soft ground treatment method to minimize the impact due to the disturbance or to take special measures to prevent serious settlement after opening to service. It is therefore necessary in the investigation stage to have an understanding of the distribution of accurate layer thickness, consolidation properties, sensitivity ratio, etc.

Peat ground

Peaty ground consisting of highly organic soil (Pm) is generally divided into peat ground consisting of peat (Pt) (w_n > 300%) with a large percentage of unresolved fiber, and muck (Mk) consisting of highly resolved organic soils. Among these grounds, peat ground does not cause stability problems if careful works are conducted, because of its high water permeability. Although muck ground has a smaller water content (w < 300%) compared with peat ground, once it is disturbed, the ground tends to suffer a serious reduction in strength; recovery of strength is slow because of its small water permeability, so it can often cause stability problems. Therefore, it is necessary to clarify whether the peaty ground is peat or muck in the investigation stage.

Ground composed of peat and marine clay

In case a thick clayey layer ($w_n > w_L$) exists under a peat layer, the minimum safety factor for the slip circle passes the bottom of the peat layer in stability calculations. However, most actual slip surfaces pass in the clay layer under the peat layer, since the consolidation speed of the peat layer is high, and an increase in strength can be expected at an early time. This type of ground failure was found frequently in Japan. A high degree of residual settlement is induced if the clayey layer is thick. Therefore, it is necessary to make a careful examination of the peat layer as well as the clayey layer under it.

2-1-2 Procedure of Investigation and Study

(1) Preliminary Investigation

A preliminary investigation for road earthworks involves clarification of the topography, geology, soil properties, and other properties of the whole investigation area, and comparison of candidate routes from the macro viewpoints of major obstacles and construction costs. It is executed to obtain data to select the best route (preliminary road design). The purpose of a preliminary investigation is to study candidate routes by comparison. Therefore, precise study is not required at this stage. In a preliminary investigation, it is important to clarify whether the ground is soft or not where the proposed route passes, and, if it is deemed to be soft, to study the scale and soil properties in general. Even if the preliminary investigation is implemented precisely, if the investigated route is not selected, all the studies become a waste of effort. Therefore, paper study and field reconnaissance are recommended as methods for a preliminary investigation. A paper study includes the collection and arrangement of data on the items listed below in order to roughly determine the spread (status of distribution) of the soft ground, the thickness of the soft soil, the type of soil, and rough estimation of the soil properties.

- The topography of the proposed area and the general condition of the geology and soil properties.
- The distribution status of soft ground.
- Past record of construction projects and past record of accidents or problems in the nearby area.
- Prohibitions or restrictions to ensure environmental preservation.
- Other factors.

(2) Preliminary Study

In a preliminary study, the following should be examined for each route in order to decide the necessity of soft ground treatment works.

a. Ground conditions such as the thickness and distribution of the soft ground, the types of soil layers, and engineering properties based on the information of other candidate routes for comparison;

b. Road conditions including the embankment height and structures based on the longitudinal alignment of the route;

c. The conditions of the surrounding area including roadside structures.

Each proposed route is studied for whether or not it complies with the prescribed road conditions, based on rough calculations about critical embankment height and total settlement for each route. Regarding the possibility of liquefaction in an earthquake, a preliminary decision is made based on the detailed topographical classifications shown in Table 3-14 by using the methods to be explained later.

Provided that improvement work is required, the type and scale of the measures are studied within the range of supposed conditions, and the preliminary cost and period are calculated based on the results of this study. The data to be obtained from these processes are utilized to compare the candidate routes and to determine the appropriate route for the proposed road. It is important to avoid a soft ground that might cause problems during the works. A non-feasible plan can cause problems up to the maintenance stage, so this point requires sufficient attention.

The results of the preliminary study are handed over to the basic investigation process together with the preliminary investigation results, and utilized to develop an investigation plan for the efficient implementation of a basic investigation. In case the soft ground has not been found in the preliminary study, an investigation for soft ground improvement can be canceled, and the study of each earthwork structure can be started based on the respective guidelines.

(3) Basic Investigation

A basic investigation is carried out after the proposed route is decided. Its purpose is to obtain the data and information in the basic road design necessary for determinations about the road, such as an outline of the structural type or dimensions of the road. In the basic investigation stage, procuring the right-of-way has generally not yet been completed. Therefore, the investigation mainly involves collecting and arranging existing data and conducting field reconnaissance. However, if there are places with the potential to seriously affect the road structure or construction cost, it is appropriate to use nearby public land as much as possible to conduct more detailed investigations, such as geophysical exploration, sounding, or boring.

The necessity of soft ground treatment work should be re-evaluated based on more detailed information, backed up by an improved understanding of the geophysical conditions and properties. This includes the distribution and thickness of the soft ground along the proposed route and the engineering properties of the soft soil. If it is decided that improvement work is necessary, the applicable improvement work methods are selected and identified according to the reasons why the work is necessary. Settlement, stability, liquefaction, and other factors are analyzed in this stage using the methods explained later. Some treatment methods are selected from among the various available methods, based on the followings;

- Conditions of the soft ground,
- Type or structure of the proposed earthworks on the soft ground
- Length of the work period.

Selecting the right methods is a great effect on the safety of the work, and also on the work cost and period. Therefore, a basic investigation and study are extremely important, and should be carefully conducted.

(4) Basic Study

According to the procedure for a preliminary study, the first step is to analyze a case in which an earthwork structure required by a road design is constructed on the original ground without any soft ground improvement, based on the results of the basic investigation or study and the ground information obtained from the basic investigation. Next, necessary of the soft ground improvement work is evaluated. Based on the ground conditions determined in the preliminary investigation, simple calculations of stability and estimations of total settlement and residual settlement are carried out in order to confirm the possibility of a safe earthwork construction on the original ground without any improvement, and the possibility of settlement after the completion of paving.

If the stability of an earthwork structure during the works is not ensured, if an excessive amount of settlement is expected, or if the remaining of harmful settlement after the completion is expected, then appropriate soft ground improvement work methods should be selected, and the rough costs and period of improvement work should be calculated, and the implementation of a detailed investigation should be continued. In addition, detailed design of an earthwork structure to be built on the soft ground, selection of appropriate improvement methods, and design of the selected method should be conducted according to the following chapters. If it is predicted that the earthwork structure is stable and that settlement will not be harmful after construction, the study process can proceed to the next stage based on the appropriate earthwork structure guidelines without a detailed investigation of soft ground.

(5) Detailed Investigation

A detailed investigation is conducted along the centerline of the proposed road alignment over the entire length of the route in order to develop the detailed design of the earthwork structures to be constructed on soft ground and the soft ground improvement work. A detailed investigation mainly includes field reconnaissance, geophysical investigation, sounding, boring, sampling, soil tests, and field tests. A detailed investigation is generally conducted in two phases: a comprehensive investigation that covers the entire route, and a local investigation that specifically focuses on problem locations or locations that require special analysis.

The first step is an investigation along the entire route at fixed intervals. Locations with drastic changes in the longitudinal or transverse direction or locations where large earthwork structures are to be constructed are then explored in more detail and at shorter intervals, in order to obtain the right amount of information necessary to analyze improvement work (e.g., the thickness of soft layers, the stratification status of the soil, physical properties or strength, consolidation properties, or liquefaction resistance). This investigation is not made up of standardized content items. It includes the major check items necessary to carry out the design or implementation of the improvement work methods identified by the basic study. Based on the information obtained from the detailed investigation, the stability of the proposed earthwork structures or the ground on which these structures are to be built is verified, and the design of the earthwork structures, the type or scale of the soft ground improvement work, or the construction specifications are ultimately determined. If the detailed investigation fails to provide sufficient information, an additional investigation is conducted.

(6) Additional Investigation

While a detailed investigation is conducted along the entire length of a route, an additional investigation is conducted concentrating on a specific location in the soft ground area. An additional investigation is conducted if the detailed investigation finds that a certain soft ground area has many issues and sufficient data for the design cannot be obtained from the results of the detailed investigation alone. Specifically, it is necessary to conduct an additional investigation in the cases noted below.

a. The position, type, or scale of a proposed road is changed.

b. The continuity of soil layers cannot be clear because the soil composition is more complicated than expected.

c. The depth of a soft soil layer varies from location to location, the slope of the foundation locally changes, and there are concerns about uneven settlement or instability of the road.

d. Samples at necessary points are insufficient, and the soil properties of the ground cannot be determined.

e. If the applicability of the selected soft ground improvement work is not ensured.

Necessary investigations and tests are added depending on the type of improvement work selected in the design stage. Some examples of these investigations and tests are shown below.

- Sand mat method: Particle size analysis or water permeability testing of sand mat materials
- Vertical drain method: Quality testing of drain materials (permeability or clogging) and ground permeability testing
- Surface and deep mixing method: Physical and chemical testing of soil, mix proportion testing, hexavalent chromium elution testing (in case cement and cement-based stabilizer are used)

f. Implementation of the selected improvement method is likely to affect the flow of groundwater

Necessary investigations and tests are added depending on the type of improvement work method selected in the design stage. Examples are shown below.

- Groundwater level lowering method: Ground permeability testing (permeability and groundwater level in the surrounding ground)
- Chemical feeding method: Feeding testing, water quality inspection, water permeability testing, flow direction and flow rate measurement, and hexavalent chromium elution testing (in case cement-based chemicals are used)
- Continuous wall improvement conducted using the deep mixing method, etc.: Pumping testing, permeability testing, flow direction/rate measurement, and water quality inspection

In case the scale of the soft ground is large, there is a small amount of work experience in similar ground, or such preconditions are estimated to have a great impact on the construction cost, it is effective to carry out trial construction of a full-size model prior to full-scale construction, to make a comparative study of the theoretically estimated prediction values and the actual behavior, and to reflect the results in the design and construction work and maintenance. The Detail of the trial construction is described in “3-2-8 Trial Construction and Investigation during Construction Stage”.

2-2 Soil Investigation

2-2-1 Study on Existing Material

This section explains existing data that are effective in discovering the presence of soft ground and the ground information that can be obtained with these data.

Topographic maps

Topographic maps with scales of 1/25,000 and 1/50,000 are commercially available; 1/2,500 scale topographic maps are also commercially available for various parts of the country for cities and their surrounding areas, and 1/5,000 scale topographic maps for other areas. Recently maps are being digitalized, and digital maps are available as well as paper maps. Soft ground is typically distributed in alluvial lowlands, and typical topographic features generally include areas with a gently sloping ground surface, which are often found as paddy fields and low marshlands. See Table 1-1 for typical areas that often appear as soft ground.

Aerial photos

Photos can be used together with topographic maps to find soft ground. Specifically, the contrast, hue, and stereoscopy allow us to read the topography, and especially the fine details of the topography. While aerial photos also express the same major topographic features of soft ground as explained in "1) Topographic maps", photos can more precisely express their details, which helps us to easily obtain the overall extent and continuity of the topography. In color aerial photos, areas with a dark brown ground surface or dark green areas are likely to indicate soft ground with high water content, because of the presence of vegetation unique to marshland. Also, cohesive soil generally appears as dark grey.

Geologic maps

These maps can be used to evaluate the type of subsurface soil or soft ground. Engineering properties of soft ground may be suggested by the origins of the ground. Geologic maps, however, are generally small in scale and therefore are often not suitable for an investigation of a small area.

Old maps, topographic classification maps, and land use maps

From old maps, we can understand the history of the ground, such as past embankments or reclamation works, and topographic features (natural dams and old river channels) from before alterations due to construction. Topographic classification maps and land use maps tell us the current topography (low marshland, flat land, embankments, or plateaus) and land use (paddy fields, upland fields, or forests). Soft ground is often used as paddy fields or fields for lotus root crops. Since urbanization prevents identification of the original topography or land use status, it is necessary to check the original land use in old topographic maps and aerial photos.

Results of existing soil investigations

If it is possible to obtain reports on past soil investigations for the subject area, they can provide useful information. However, since soft ground has a complicated soil composition and considerable variation in its soil properties, even if existing data are used, it is necessary to check if the ground condition of the existing investigation is similar to the current subject area. Moreover, current databases containing soil investigation result data may be available, and it is good to use such databases as reference data.

Work records of other construction projects in the area and records of experimental construction work

Construction records of other construction projects or records of trial construction work conducted in the vicinity of the subject area include problems in the design and construction and how they were solved or handled. This can often provide useful information for new projects. However, unlike existing soil investigation results, they are generally not records on the proposed route of the subject road. Therefore, they must be checked together with other data.

Records of changes and disasters

Records of investigations on settlement and other problems of earthwork structures located in the vicinity of the subject area or records on liquefaction of the ground in an earthquake can help to clarify the problems of soft ground. Interview investigations with residents can also be effective.

2-2-2 Field Reconnaissance

A field reconnaissance is an examination to be conducted to clarify the condition of soft ground together with document-based investigations conducted beforehand. It can solve questions raised from reading topographic maps and aerial photos and confirm the topography. For the field reconnaissance, the existing information such as disaster record, cameras, hand levels and measuring tape are useful. The following points require attention for conducting a field reconnaissance.

a. Evaluate topographic maps or aerial photos comparing with the actual topographic features.

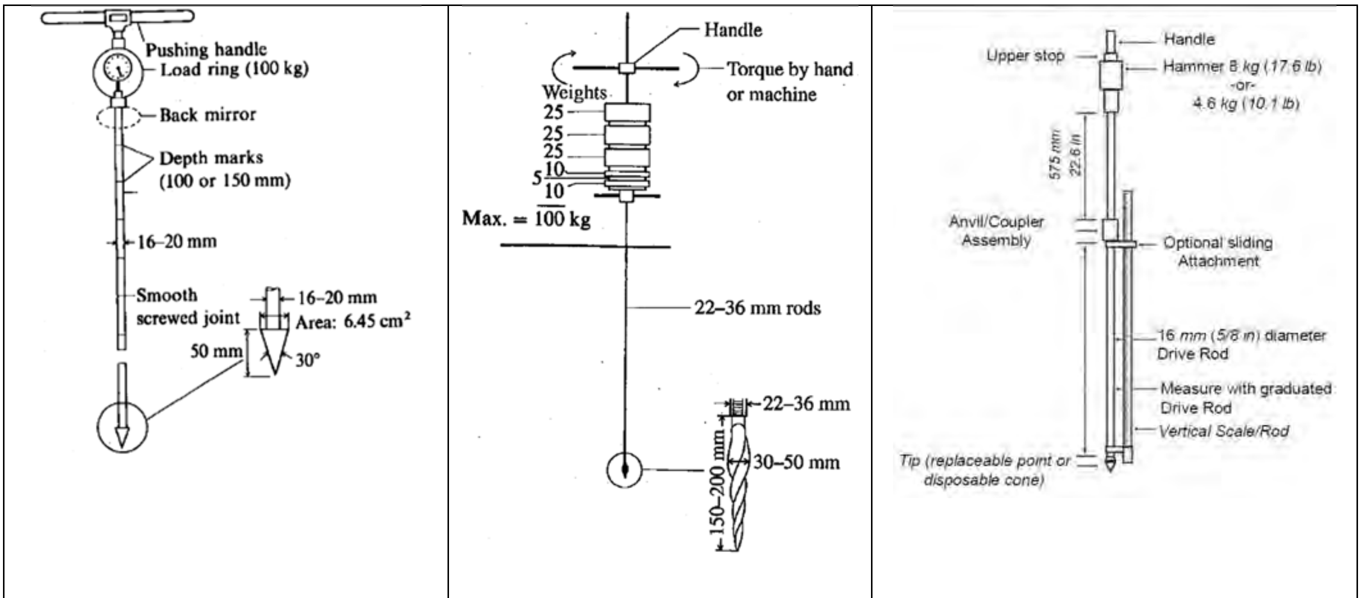
b. Make a detailed examination of the condition of the existing earthwork structures in areas that are assumed to be soft ground. For road construction, the check items may include:
   - embankment height,
   - flatness of the road surface,
   - the status of vertical undulation of the roadside ground,
   - settlement or meandering of side gutters at the slope toe,
   - settlement or movement of a structure crossing the road,
   - cracks, and
   - level differences on the road before and after the structure.
   - the lines of utility poles and the condition of houses

- agricultural water channels

2-2-3 Sounding

(1) Simple Sounding

The strength of soft ground is often estimated by means of a cone penetration test (portable cone penetration test, Dutch double-tube static cone penetration test, or electric cone penetration test) or the Swedish weight sounding test. Portable cone penetration test (Figure 2-1 a) and Swedish weight sounding test (Figure 2-1 b) are sometimes used as simple sounding investigations. Also Dynamic cone penetrometer (Figure 2-1 c) is sometimes used as simple sounding investigations. Portable cone penetrometer is suitable for shallow soft clay layers, and Swedish weight sounding test is suitable for shallow sandy layers. Dynamic cone penetrometer test is used for deeper and harder soil than the layers of portable cone penetrometer and Swedish weight sounding applicable. They should be conducted in at least one location in an area that can be evaluated as sharing the same soil composition as the subject ground. If a soft ground area spreads over a long distance, the number of test locations are increased as appropriate.



a. Portable cone penetrometer (Handheld penetrometer) 1)

b. Swedish weight sounding 1)

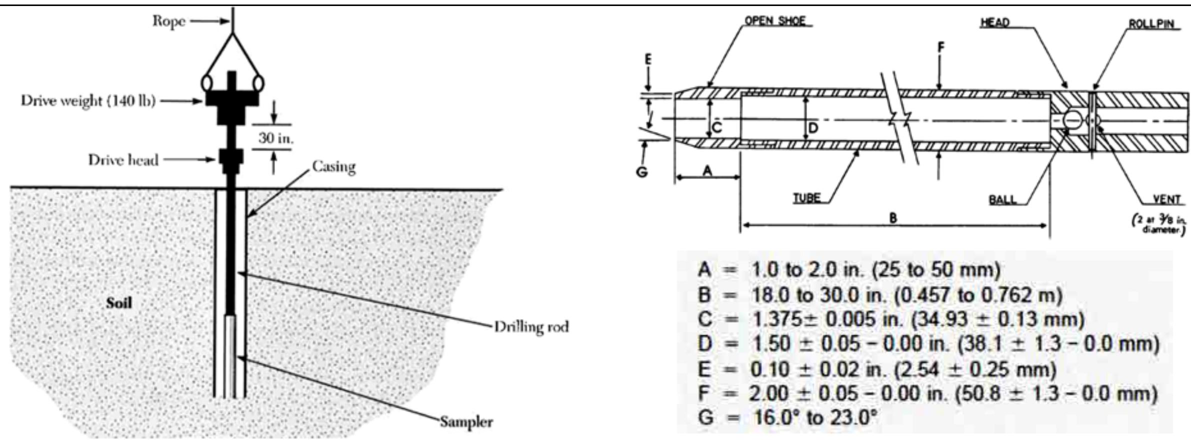
c. Dynamic cone penetrometer (ASTM D6951)

Figure 2-1 Simple sounding equipment

(2) Sounding

Sounding is conducted to clarify the thickness of soft ground, the soil layer conditions, the presence of intermediate sand layers, and to obtain data for a soil classification evaluation prior to design development. The specific process involves: (a) inserting a resistive element into the ground, attached to the tip of a pipe or rod; (b) applying various forces (e.g., penetration, rotation, and extraction); (c) measuring the resistance of the soil against these applied forces; and (d) evaluating the relative distribution and strength of the soil layers. Sounding is usually carried out prior to boring or sampling, and the results of sounding tests are used to determine the location and depth of sampling. In Japan, popular sounding tests include the standard penetration test, Swedish weight sounding test, and cone penetration test.

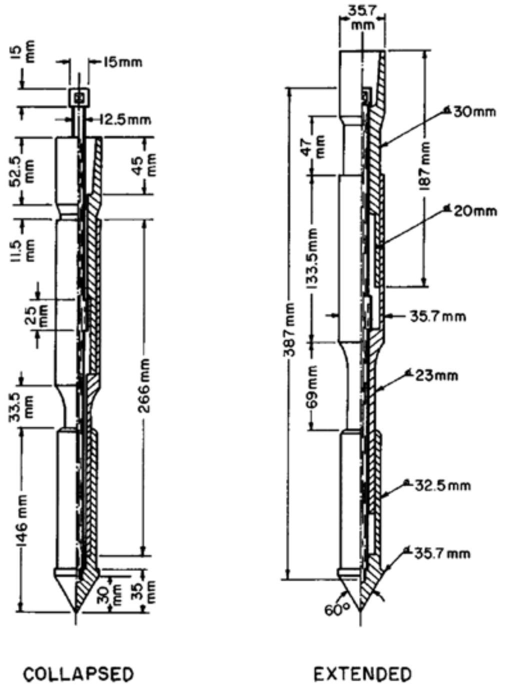
Because it uses a borehole, an important advantage of the standard penetration test is that it is possible to continue testing (N-value measurement) and obtain soil samples despite encountering a hard soil layer in the ground. In general, a standard penetration test is conducted for every one-meter depth. To be on the safe side, the recommended test depth is the depth of a soil layer that provides sufficient bearing capacity regardless of the type of soil layer. N-values are used to evaluate the ground structure as well as to estimate soil properties. However, the N-value basically indicates a broad index of ground hardness, so it is poor in accuracy as an index for estimating the dynamic properties of soft cohesive soil and produces a wide range of estimation results, which requires attention. Also, when the N-value is used to study the possibility of liquefaction of a sandy soil layer, note that correction is necessary, using the vertical load, the fine-grained content, the coefficient of plasticity, etc.

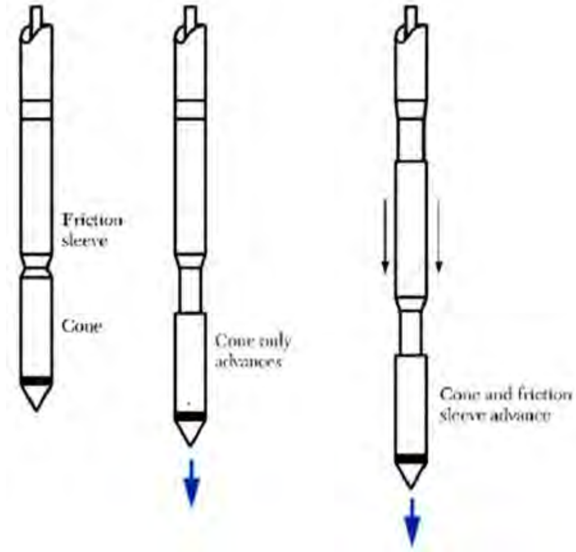


Schematic Model of SPT [2] Sprit-Barrel Sampler for SPT *(ASTM D1586)*

Figure 2-2 Standard penetration test (SPT)

Mechanical cone penetration test, often called the Dutch cone since it was first developed and use in The Netherlands, operates using a telescoping penetrometer tip, resulting in no movement of the push rods during the measurement of the resistance components. Design constraints for mechanical penetrometers preclude a complete separation of the end-bearing and side-friction components. The Dutch double-tube static cone penetration test can penetrate an intermediate sandy layer of a certain level of strength and provides continuous soil data.





Example of a Mechanical Friction-Cone *(ASTM D3441)*

The lower part of the tip, including a mantle to which the cone attaches, advances first until the flange engages the friction sleeve and then both advance.

Operation of Mechanical Friction-Cone [2)]

After the penetrometer is at test depth, thrust on the inner rod advances teh cone for 30mm. Next teh cone and teh friction sleeve are advanced together for readings of cone bearing and soil friction.

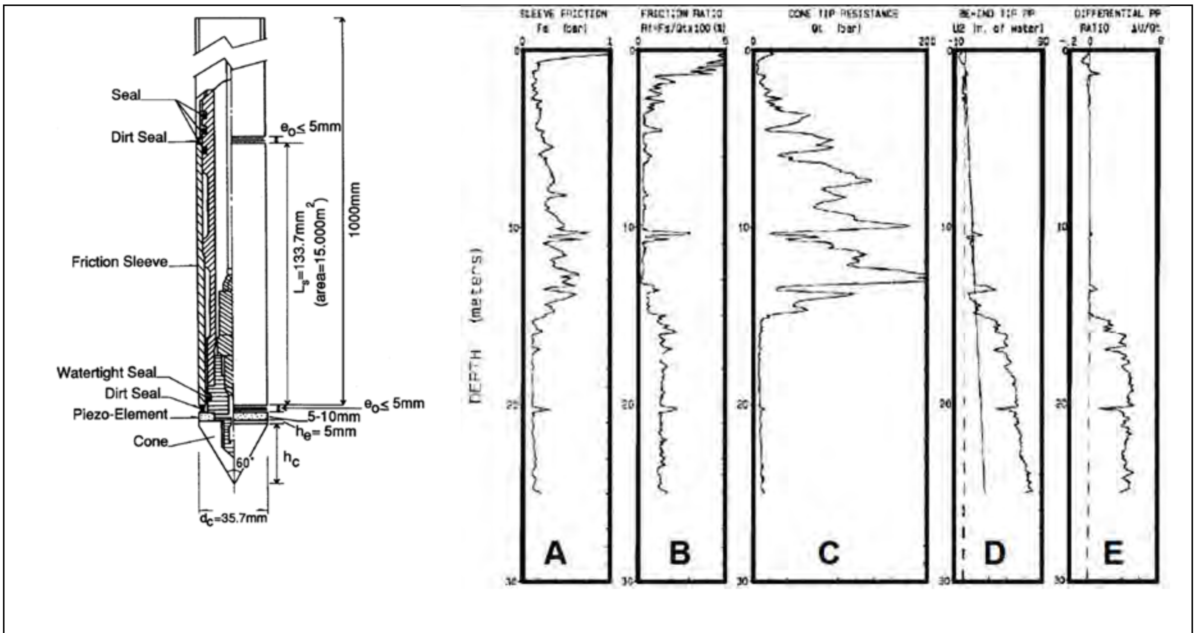
Figure 2-3 Examples of cone tips of mechanical cone penetration test

The electric cone penetration test can additionally measure pore water pressure, thereby estimating soil properties or confirming the presence of a draining layer. With the Swedish weight sounding test, it is possible to explore a large number of locations, as it is easy to operate and requires a short time for investigation.

Rig with Anchoring

Electric Cone Tip

Figure 2-4 Examples of electric cone penetrometer appratus [9)]



Cone Tip

Example of CPT Results
A: Sleeve Friction, B: Friction Ratio, C: Cone Resistance,
D: Pore Pressure, E: Deferential PP Ratio

Figure 2-5 Examples of cone tip and test results of electric cone penetrometer test *(ASTM D5778)*

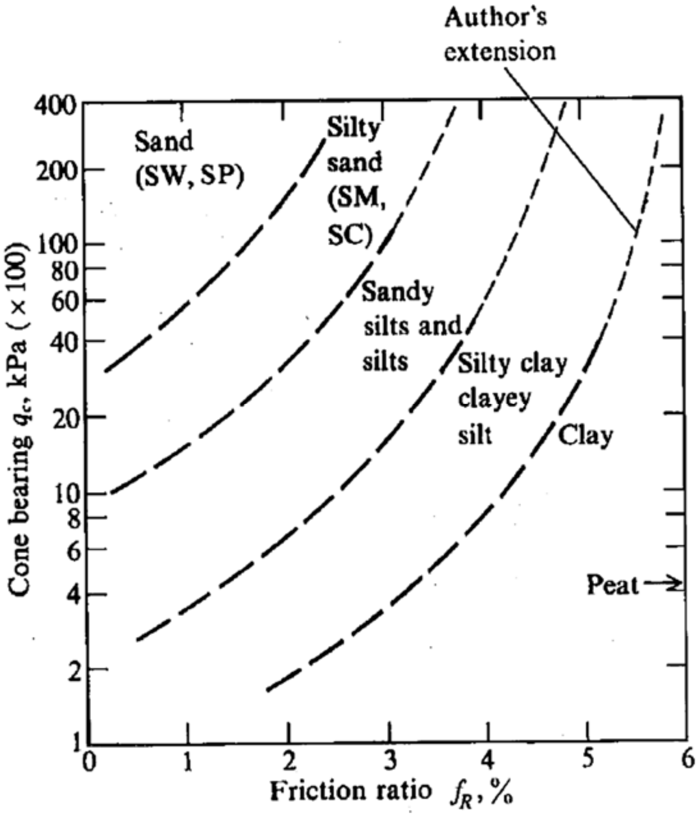


Figure 2-6 Soil classification charts using a electric or mechanical cone [1)]

The sounding interval needs to be changed depending on the topography or the degree of changes in strata. The typical interval along the road centerline is from 20 to 100 m. The recommended investigation procedure is to conduct initial tests at intervals of about 100 m. The results of these tests are then used to increase the number of investigation locations at intermediate points, etc. Where topographic changes are drastic or the soil composition of soft ground is complicated, sounding is conducted even at the toe of the slope on both sides of the proposed embankment in order to clarify changes in soil composition in the transverse direction.

In particular, if a route crosses soft ground in a valley area, the thickness of the soft ground often drastically changes in the longitudinal direction of the road, and other geotechnical changes are often seen in the transverse direction of the road (e.g., the slope of the foundation, or changes in the thickness of the soft soil layer). Therefore, in order to grasp the overall geometry of the foundation and its soil properties, it is necessary to explore the ground along the centerline of the road as well as the area around the toe of the slope on both sides of the proposed embankment by carrying out sounding tests, etc. As a result of this process, a soil profile is prepared in the longitudinal direction of the road as shown in Figure 2-37. Therefore, it is recommendable to carry out sounding tests at two or more locations in the longitudinal direction of the road within a range that is topographically assumed to be a single soft ground area. Table 2-2 shows the properties of major sounding.

Table 2-2 Characteristics of sounding methods and field tests

	Name	Measurement Value	Value can be estimated	Applicable ground	Reachable depth	Note
Static	Swedish weight sounding test	Load with 1000 N or less (Wsw), no. of half turn per penetration of 1 meter (Nsw)	Converted to N-value or shear strength (various equations proposed)	Clayey soil Sandy soil	about 10 m	Operation easier than standard penetration test
	Portable cone penetration test	Cone resistance qc	Shear strength	Clayey soil Organic soil	about 5 m	Simple test and quickly completed
	Dutch double-tube cone penetration test	Cone resistance q_c	Shear strength	Clayey soil Sandy soil	Depends on the capacity of the device	High reliability
	Electric cone penetration test	Cone resistance q_t Pore water pressure u skin friction f_s	Shear strength Soil classification Drainage and consolidation characteristics	Clayey soil Sandy soil	Depends on the capacity of the device	High reliability
	In-situ vane shear test	Maximum torque	Non-drained shear strength of cohesive soil	Soft clayey soil	about 15 m	Dedicated to soft clayey soil; directly measures c_u
	Borehole lateral load test (Pressure-meter test PMT)	Pressure, Borehole wall displacement	In situ modulus of deformation (modulus of elasticity, in situ overburden pressure, yield pressure)	All types of soil and rock (borehole wall must be stable and smooth)	Basically no restriction	Soil mechanical values can be obtained theoretically
Dynamic	Standard penetration test	N-value (number of blows)	density of sand, internal friction angle, modulus of elasticity liquefaction strength, bearing capacity, unconfined compressive strength	All types of soil except for cobble or boulder	Basically no restriction	Applicable for all kind of ground
	Simple dynamic cone penetration test	N_d (number of blows)	N_d = (1–2)N Same concept as N-value	All types of soil except for cobble or boulder	about 10 m (rod friction increases with depth)	Operation easier than standard penetration test

2-2-4 Investigation Boring

As one of the most important methods of soil investigation, boring is frequently used in each stage from a preliminary investigation to a detailed investigation.

With this method, soil properties and soil composition can be roughly determined from the drilling resistance, the drilling speed, and the slime in the drilling fluid. In addition, sampling and in situ tests are usually conducted in boreholes. Since borehole diameters vary depending on their purpose of use, it is necessary to determine an appropriate diameter. Table 2-3 summarizes the purposes of use and hole diameters of boreholes.

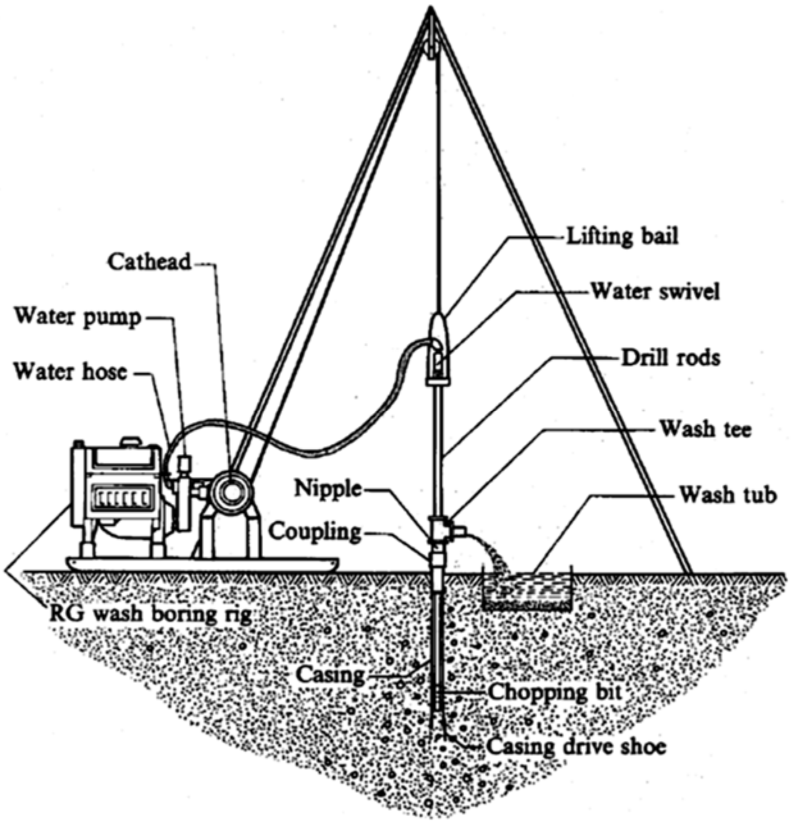


Figure 2-7 Schematic of wash-boring operations [1)]

Figure 2-8 Rotary boring machine belong to SRL [(TCP)]

Table 2-3 Use purposes of boreholes and borehole diameters 1)

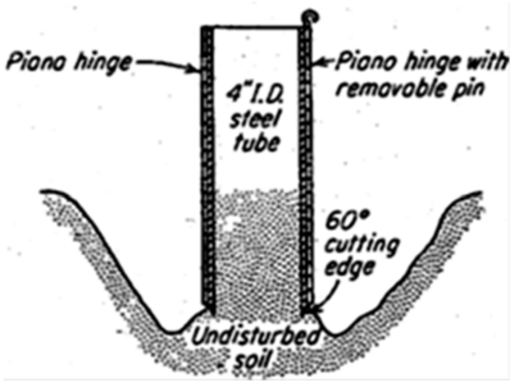
Test and sampling in borehole	Borehole diameter (mm)	Purpose				
		Soil distribution	Embankment stability	Embankment settlement	Lateral deformation	Liquefaction
Standard penetration test (SPT)	66	++	++	+	+	++
PS logging	66-116	++	—	—	—	+
Sampling (piston sampler)	66-116	++	++	++	++	+
Sampling (double-tube / triple-tube sampler)	116	++	++	++	++	++

++- Suitable, +- Applicable

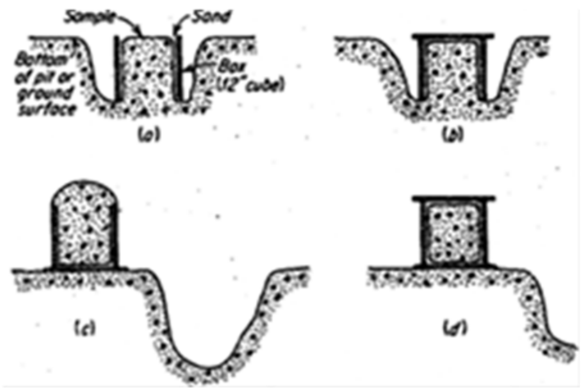
2-2-5 Sampling

(1) Simple Sampling

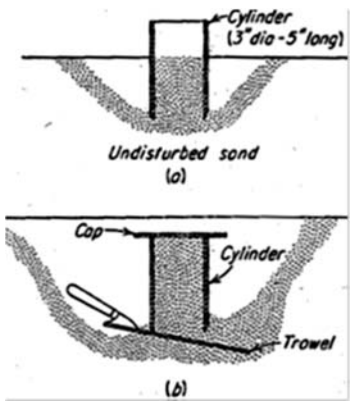
Soil samples taken from the surface are used for soil laboratory tests to classify the soil and obtain the soil properties. In addition to borehole sampling in soils, undisturbed samples may be taken by hand from naturally or artificially exposed earth surfaces from in excavated trenches and pits as shown in Figure 2-9. For disturbed sampling, scoops and hand augers as shown in Figure 2-10 are used.



Surface sampling in cohesive soil without gravel or pebbles



Surface sampling in cohesive soil with gravel and pebbles



Surface sampling in sand

Figure 2-9 Examples of surface undisturbed sampling 5)

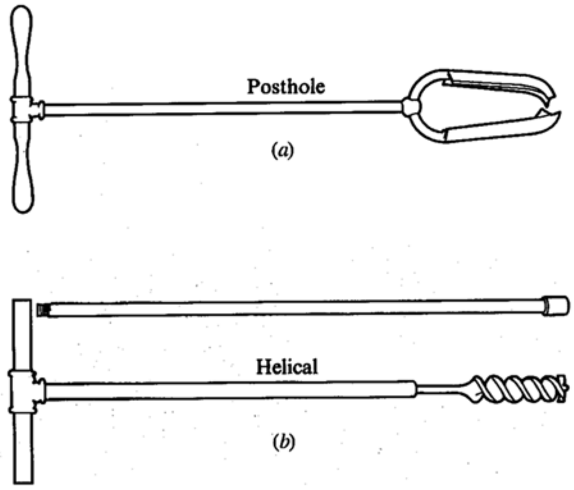


Figure 2-10 Hand augers used for disturbed sampling [1)]

(2) Sampling with Investigation Boring

Undisturbed samples should be taken from the ground using an appropriate sampler in order to properly conduct strength tests or consolidation tests of the soils in soft soil layers formed in the ground. Appropriate sampling locations are selected based on the results of the sounding tests. Mainly locations with a thick soft soil layer or the smallest strength are chosen. In the case of sandy soil, locations considered to have liquefaction potential should be selected. The standard sampling interval in the longitudinal direction of the road should be 50 to 100 m. However, in an area that is topographically assumed to be a single soft ground area or in an area that crosses soft ground in a valley, it is recommendable to carry out sampling at two or more locations at least in the longitudinal direction. Sampling is generally conducted along the centerline of the road. For stability analysis of a high embankment, sampling is conducted near the slope toe of the embankment. In particular, sampling is carried out at the foot of the slope if a soft soil layer is found at the slope toe of a high embankment. In principle, a borehole for sampling should reach the bearing layer under the soft soil layer. It is not generally necessary to take samples from all layers where the ground has no possibility of liquefaction, a low embankment that requires no analysis against slip failure is designed, and there is a thick soft soil layer. Sampling may be stopped at about half the depth equivalent to the width of the road site. However, this is based on the condition that the thickness of the soft layer is confirmed by boring, sounding, or other means.

The minimum condition for the vertical sampling interval with a sampler is that samples should be taken from at least one location from each layer of soil considered to be a single soil type. When such a layer is thick, sampling from the same layer at more than two to three locations is recommendable. The sampling depth should be determined based on the sounding results. Note that since soft ground is often characterized by complicated changes in soil properties in the depth direction, it is difficult to clearly categorize soil layers. Therefore, it is not easy to take typical samples from each soil layer. Although it is a standardized approach, a practical solution to such a situation of very complicated changes in soil properties is to take samples every one to two meters in depth.

Various samplers have been developed to date, and some of the standard types are listed in Table 2-4. This table shows a detailed classification of the items "Sampling with fixed piston sampler" and "Sampling with double-tube/triple-tube sampler" in Table 2-3. Each sampler is applied to certain types of soil. Particularly undisturbed sand sampling should be selected carefully because even little disturbance in the undisturbed sand sample can impact seriously on the test results. Therefore, the application of sampling requires careful study. The standard samplers listed in Table 2-4 generally have difficulty taking samples from sandy soil layers. For such soils, estimations can be made by means of formulas using the N-value, etc., mentioned in "2-3 Laboratory soil tests". However, a number of advanced sampling methods have been developed including frozen sampling that ensures obtaining a high quality sample and sampling with a polymer agent to protect the core.

Table 2-4 Applicable soil properties and characteristics of major samplers

Type of sampler		Structure	Type of ground							
			Cohesive soil			Sandy soil			Gravel	
			Soft	Medium	Hard	Loose	Medium	Dense	Loose	Dense
			N-value guideline							
			0–4	4–8	8 or more	10 or less	10–30	30 or more	30 or less	30 or more
Fixed piston thin wall sampler	Extension rod type	Single tube	++	+	-	+	-	-	-	-
	Hydraulic type	Single tube	++	++	+	+	-	-	-	-
Rotary double-tube sampler		Double tube	-	++	+	-	-	-	-	-
Rotary triple-tube sampler		Triple tube	-	++	++	+	++	++	-	+
Rotary built-in sleeve double-tube sampler		Double tube	-	+	+	-	+	+	-	
Block sampling		-	++	++	++	+	+	++	-	+
Rotary tube sampler		Multiple tube	-	-		-	-	-	-	-

**1: ++: Suitable; +: Applicable*

**2: Applicability for sandy soil has been confirmed only in terms of shear strength tests. Applicability for tests to obtain deformation coefficients or liquefaction resistance is poor.*

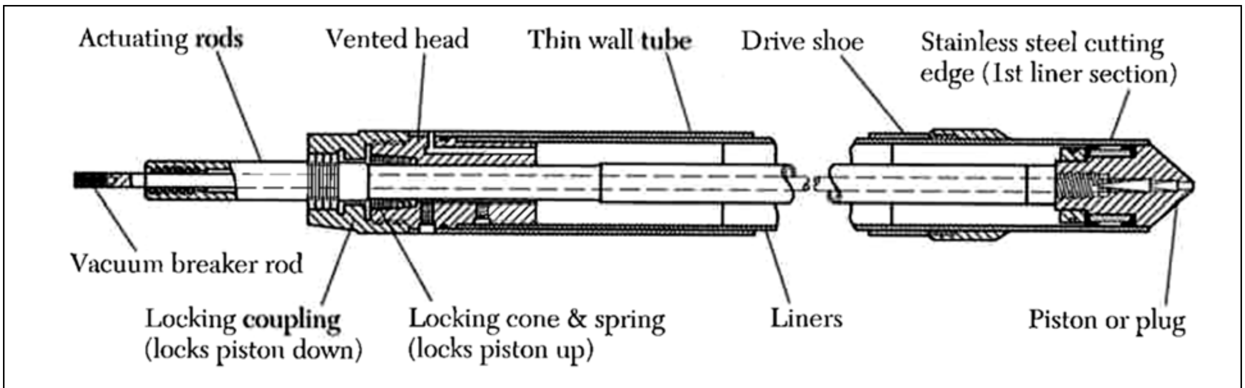


Figure 2-11 Fixed piston thin wall sampler – extension rod type [2)]

Other than samplers listed in Table 2-4, Shelby tube sampler, which have steel ball instead of piton to create vacuum, is often used for undisturbed sampling. However, quality of the sample taken by Shelby tube sampler is not so good as compared with the other samplers listed in Table 2-4.

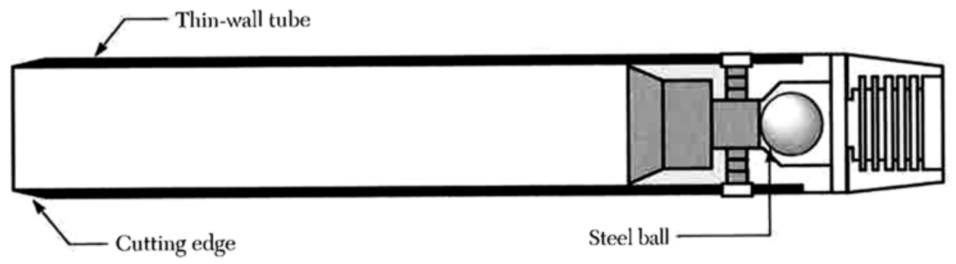


Figure 2-12 Shelby tube sampler [2)]

2-2-6 In Situ Tests

(1) In Situ Tests in Borehole

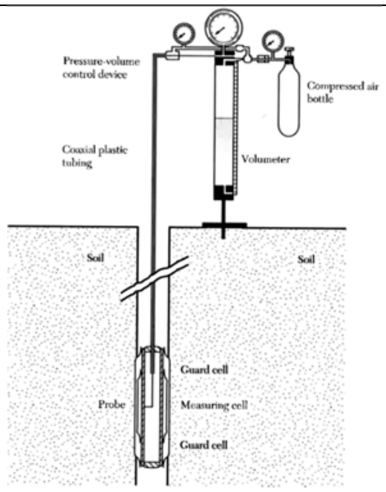
In situ tests include borehole lateral load test and vane test, as listed in Table 2-5 are designed to obtain the conditions or quality of the ground on-site avoiding ground disturbance. Borehole lateral load test, often called pressure-meter test and field vane shear test are designed to directly obtain elasticity or strength of the subsurface ground in boreholes. These tests are effective for the ground where sampling is difficult, for the anisotropic ground, for the ground such as fibrous organic soil where undisturbed sampling is difficult.

Pressure-meter test (Borehole lateral load test)

Six types of the pressure-meter test are listed in Table 2-5. Essentially, the pressure-meter consists of a cylindrical rubber membrane expanded against the sides of a borehole which is either predrilled in the case of the Menard-type pressure-meters or formed by the equipment in the case of the Camkometer and push-in types. The expansion of the membrane is measure directly by feeler gauges, or indirectly by measuring the volume of water or oil required for the increased diameter. The pressure-meter produces a pressure-volume curve of the type shown in Figure 2-14. The modulus of elasticity is obtained from the slope of the unload-reload cycle after the expansion has reached the plastic stage.

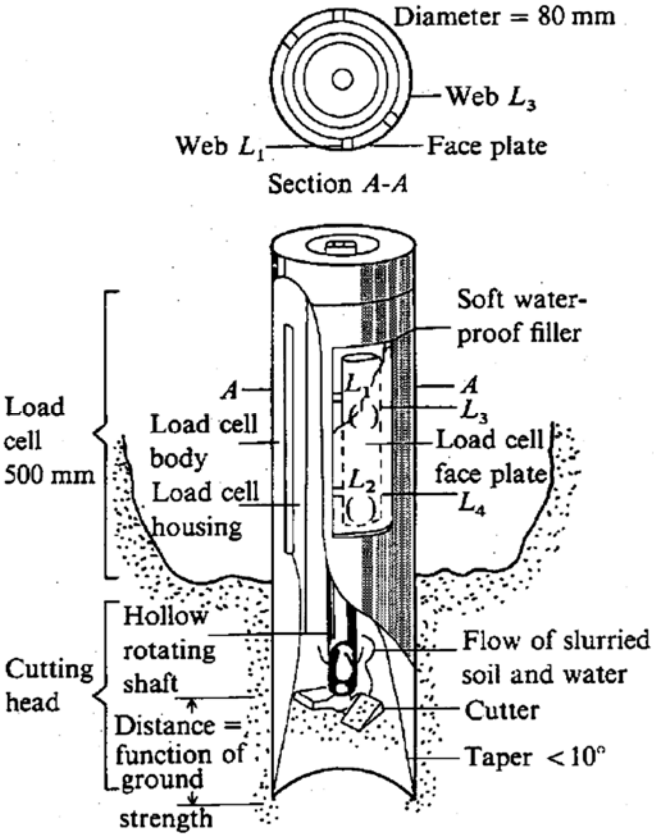
Table 2-5 Types of pressure-meter [3)]

Type	Installation method	Measurement system	Diameter (mm)
Menard pressure-meter (type GB)	Lowered into preformed hole at base of borehole	Membrane expanded by water pressure. Volume measured at surface	32, 44, 58, and 74
OYO LLT	Lowered into preformed hole at base of borehole	Membrane expanded by water pressure. Volume measured at surface	86
OYO Elast-meter (type100)	Lowered into preformed hole at base of borehole	Membrane expanded by water pressure. Expansion measured by displacement transducer	70
Self-boring pressure-meter (Camkometer)	Drilled into soil by integral unit	Membrane expanded by gas. Expansion measured by three strain gauged feeler arms	82
Cambridge in-situ high-pressure dilatometer	Lowered into preformed hole at base of borehole	Membrane expanded by oil pressure. Expansion measured by six strain gauged feeler arms	74
Building Research Establishment push-in pressure-meter	Pushed into soil at base of borehole, or into under-size pre-cored hole	Membrane expanded by oil pressure. Volume measure at surface	78



Menard Pressure-meter Type G [2)]

Low pressure type LLT [10)]



Camkometer
Self-boring Pressure-meter[1)]

Figure 2-13 Examples of lateral load test apparatuses

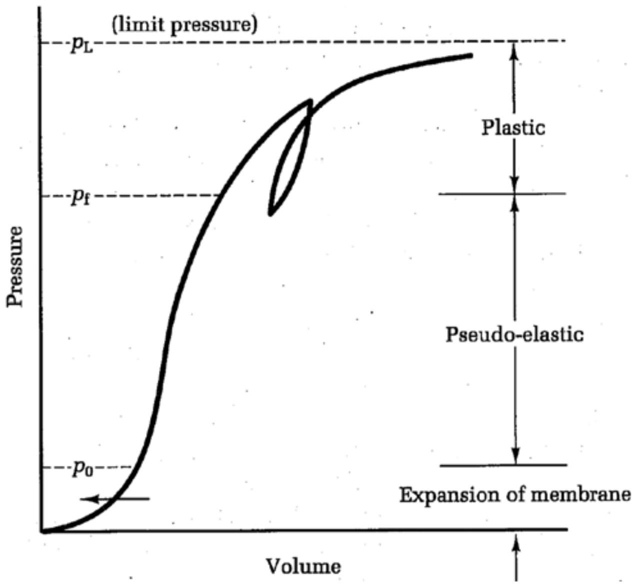


Figure 2-14 Volume (displacement)-pressure curve for pressure-meter test [3)]

Vane shear test

The vane shear test is a substantially used method to estimate the in situ undrained shear strength of very soft, sensitive, fine-grained soil deposits. This test is closely related to the laboratory consolidate-undrained shear strength test. The test is performed by inserting the vane into the soil and applying a torque after a short time lapse on the order of 5 to 10 minutes. The vane may be inserted into the stratum being tested from the bottom of a borehole or pushed without a hole by using a vane sheath similar to a cone penetration test.

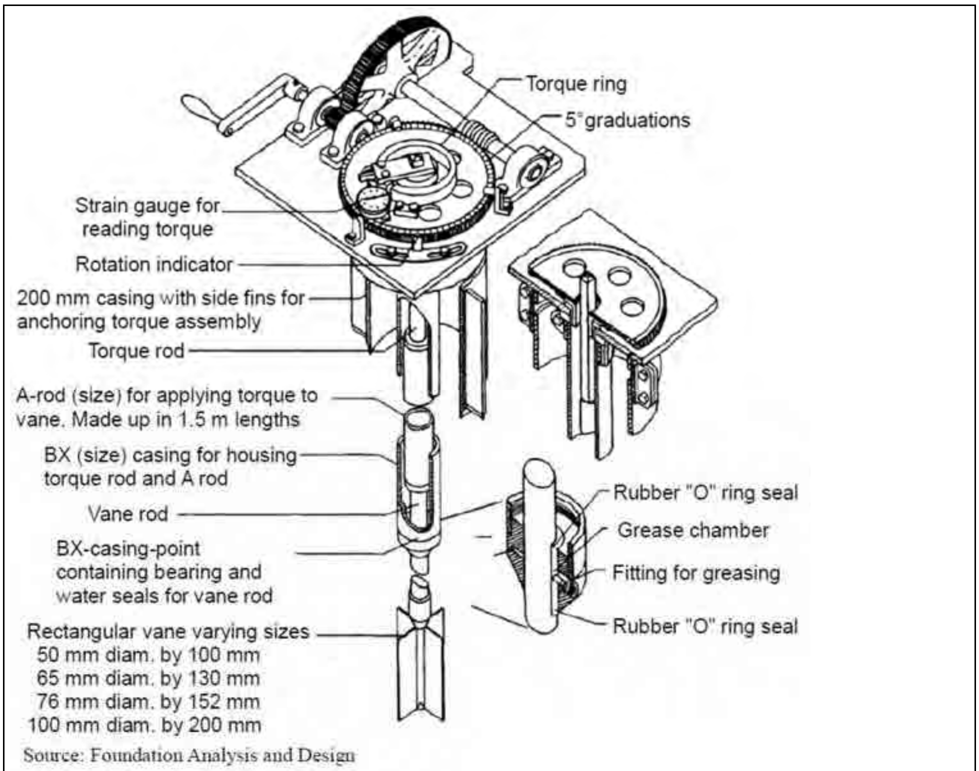


Figure 2-15 Field vane apparatus [1)]

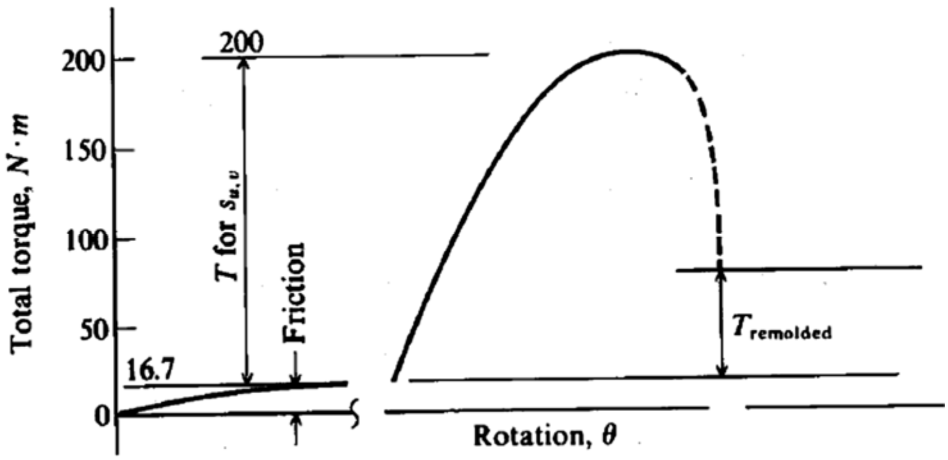


Figure 2-16 Typical vane shear data 1)

(2) In Situ Tests in Field

Obviously the most reliable method of obtaining the ultimate bearing capacity at a site is to perform a load test. This would directly give the bearing capacity if the load test is on a full-size foundation such as footing; however, this is not usually done since an enormous load would have to be applied. The usual practice is to load-test small steel plates of diameters from 0.3 to 0.75 m or squares of side 0.3 x 0.3 to 0.6 x 0.6 m. Figure 2-17 presents the essential features of the load test. Figure 2-18 is a typical plot of load versus settlement approaches the vertical, one interpolates maximum bearing capacity. The maximum bearing capacity is obtained as that value corresponding to a specified displacement (as 25mm).

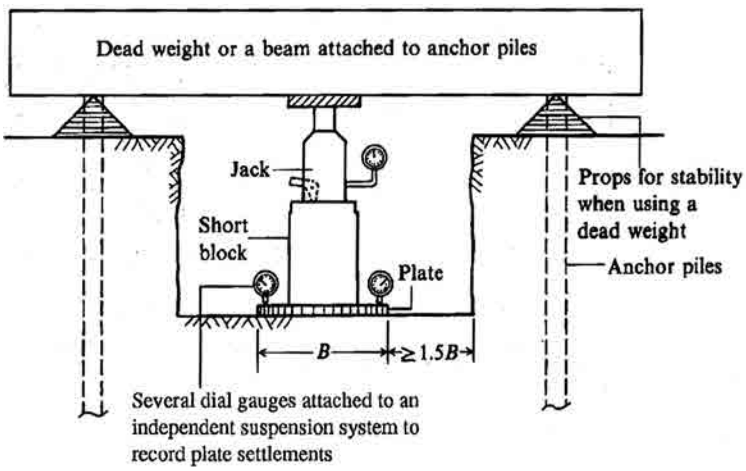


Figure 2-17 Plate load test 1)

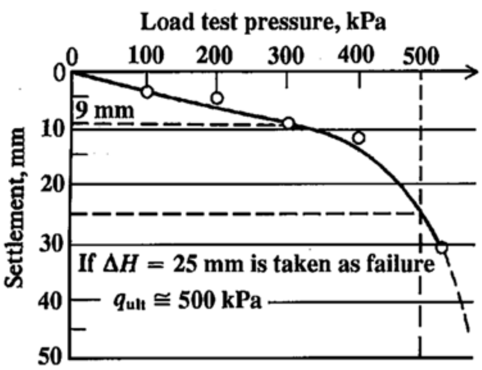


Figure 2-18 load vs. settlement plot 1)
(to establish the maximum design pressure)

Figure 2-19 shows a conceptual drawing illustrating the scope of influence of the load depending on the different area of loading. It is essential to properly understand that the plate loading test results show the characteristics of the bearing capacity of the ground corresponding to the size of the plate.

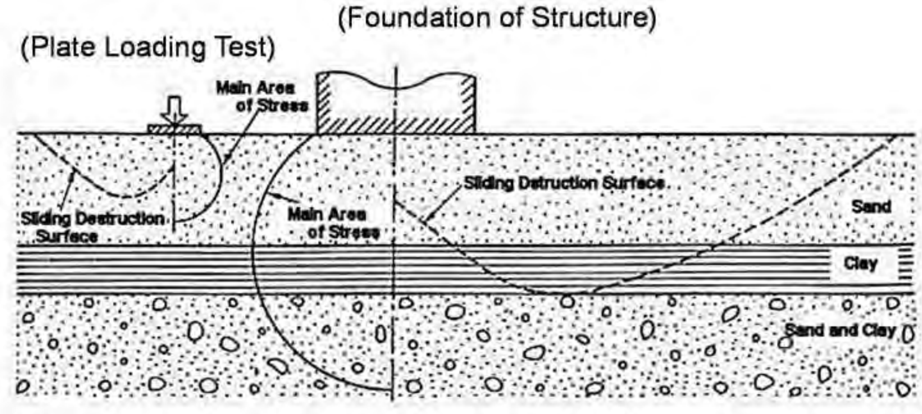


Figure 2-19 Effect of size between foundations for structure and loading plate 4)

In these ground conditions it is necessary to sink a nuber of trial pits down to or below foundation level or employ deep plate load test. Figure 2-20 show a example of arrangement of the deep load test at the bottom of a large diameter borehole drilled until the foundation level.

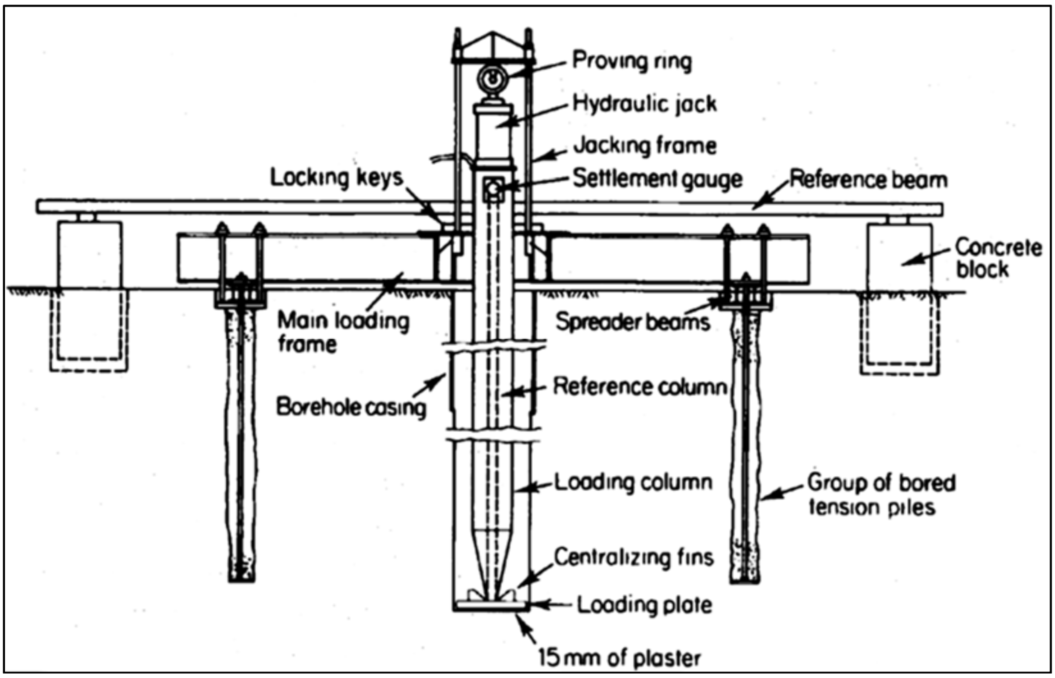


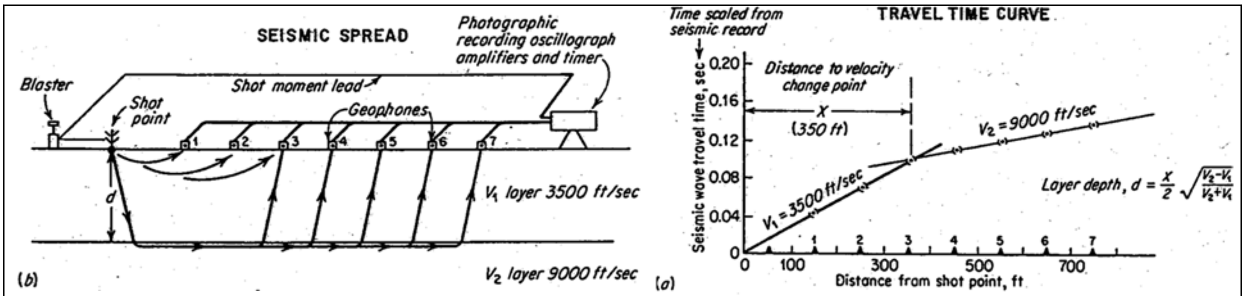
Figure 2-20 Example of arrangement of deep plate load test 3)

2-2-7 Geophysical Exploration

Geophysical exploration includes various methods using elastic waves, electricity, electromagnetic waves, or gravity. The subjects of measurement in geophysical investigation are physical quantities: they do not directly reveal any engineering properties of the ground. However, unlike other investigations such as sounding or boring, this kind of investigation can clarify the geological conditions continuously along the investigation line. With this and other advantages, geophysical investigation is effective in roughly clarifying the ground condition, or in some cases for identifying relatively vulnerable locations.

(1) Seismic Survey

Seismic surveys include refraction, reflection, and acoustic techniques. All are based on the fact that the elastic properties of earth materials determine the velocities of waves propagating through them.



A number of geopones are set in a line on the ground at measure distances apart. A few amout of explosive is exploded at the shot point. Sismic waves sent into the ground by teh explosion reach the geophohnes are picked up by the geophones.

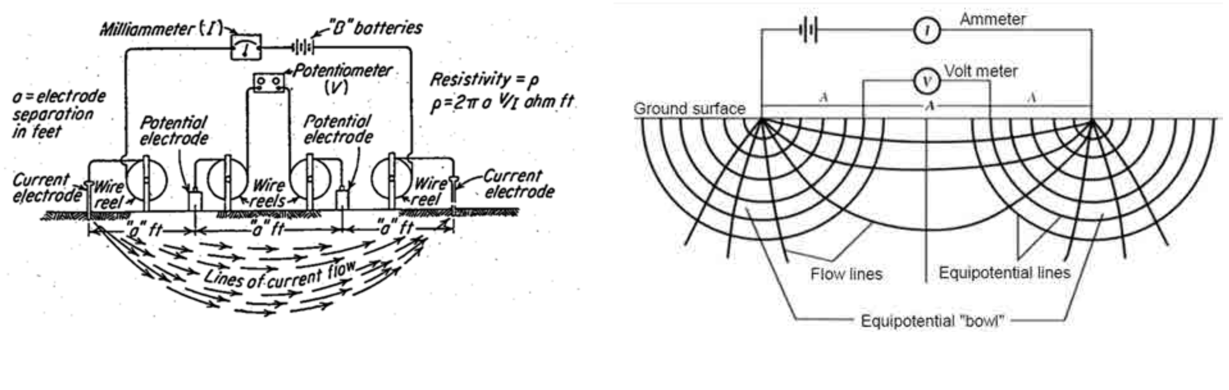
The seimic waves travel with a velocity V_1 in the top layer. In this V_1 layer, the seismic waves travel at considerably less velocity than in the underlying layer wherein they travel with a velocity V_2. Then the thickness d may be easily determined by the formula.

Figure 2-21 Seismic refraction method [5)]

As with seismic surveys offer several advantages in investigation; environment is not disturbed, the equipment is portable, and large areas can be covered at relatively small cost. However, interpretation of seismic measurements is also conjectural where the geology is complex and velocities of the various materials are not in sharp contrast.

(2) Resistivity Survey

Surface-based measurement of the electrical resistivity of earth materials involves the introduction of an electrical current into the ground and the measurement of the materials' resistance to the current. There are several variations to the resistivity survey method. All introduce a controlled electrical current into the earth materials through tow current electrodes. The resistance of the materials to the current is measured by the potential difference between two potential electrodes place within the field created by the current electrodes. The resistivity surveys can be conducted to provide vertical or horizontal profiling. In vertical profiling, the center of the electrode spread in kept fixed at a desired location. Because increased spacings result in increased depths of investigation, this procedure is called sounding. In contrast, horizontal profiling, sometimes referred to as electrical mapping, employs a constant electrode spacing with the array moving so as to center at a series of desired map locations. The major advantages of the resistivity surveying techniques lie in the portability and simplicity of the instrument. Large areas can be covered relatively rapidly at small cost. The major disadvantage is that the interpretation of the measurements is neither simple nor unique, especially in areas where the strata are not horizontal, the structures are complex, the layers are nonuniform, or contrasts in material resistivities are not great.



Schematic setting of resistivity survey

Wenner electrode arrangement with a flow net for a homogeneous soil

Figure 2-22 Resistivity survey [5)]

(3) PS Logging

PS logging (seismic velocity logging) is sometimes conducted during a detailed study of soft ground (e.g., seismic response analysis of an important structure). PS logging is a type of geophysical investigation using a borehole and provides seismic velocities of the ground (P waves and S waves). A model for seismic response analysis of the ground is determined based on these results, including establishing the shear rigidity of the ground G or the rock surface for seismic design.

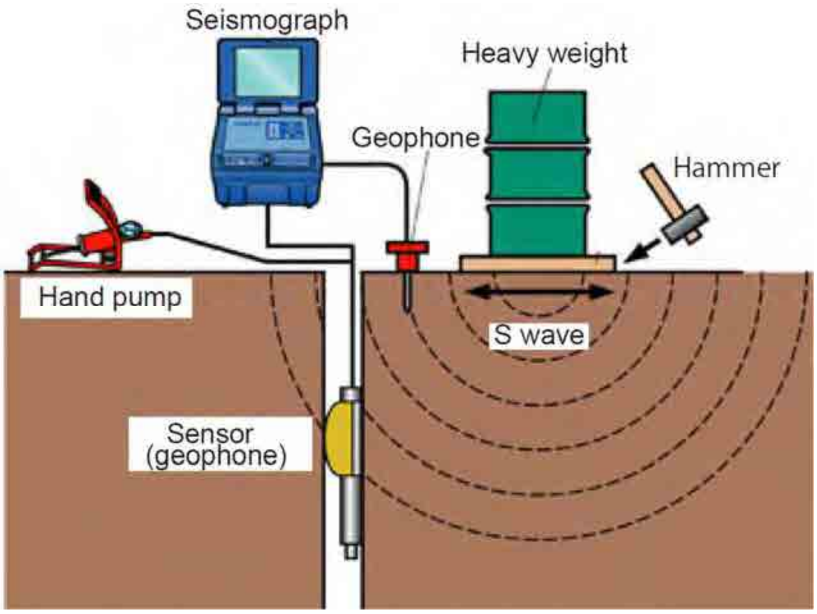


Figure 2-23 PS logging [10)]

2-3 Laboratory Soil Tests

Typical soil tests used for soil type evaluation, stability checks, compression settlement analysis, and seismic analysis are shown in Table 2-6. Special soil tests are required depending on the analytical method or analytical precision of each study. Disturbed samples are used in soil tests for evaluation and classification of soil types. For study or design related to embankment stability and settlement, as well as in an earthquake, it is necessary to use undisturbed samples, to carry out soil testing in a laboratory environment, and to obtain soil properties. When a soil test is conducted on an undisturbed sample in a sampling tube, careful handling is very important to prevent inadvertently applying vibration to the tube. Also, using a sample from the end of the tube should be avoided due to the risk of negative impacts from changing or disturbing the water content ratio.

2-3-1 Physical Properties

The following physical soil properties can be useful for the preparation of soil profile;

- natural water content
- specific gravity
- liquid limit / plasticity limit
- grain size
- organic content

Disturbed samples obtained from the standard penetration test can be used for these tests. Among these tests, the natural water content is important to clarify the differences in soil layers, and it is also deeply related to strength or deformation properties. Therefore, this test shall be done on the samples as many different depths as possible. The natural water content may not be necessarily determined independently: for example, the water content can be obtained during unconfined compression test. The distribution of water content in the depth direction at each investigation point is important. Soil properties other than the natural water content are not necessarily to be obtained for each of all the sampling tubes. Only the values that represent each layer are sufficient. If these soil properties are different, the original soil layers are also considered to be different.

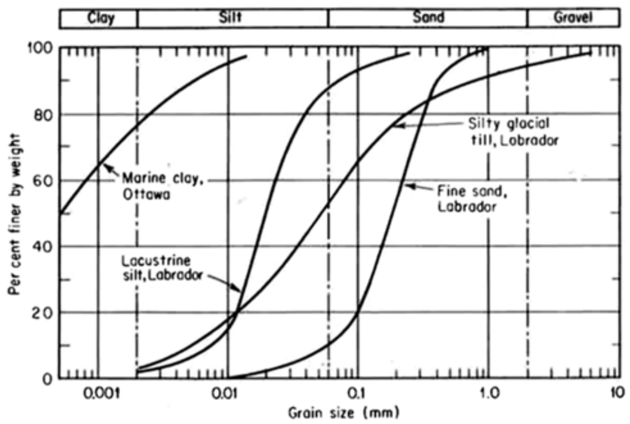


Figure 2-24 Example of grain size distribution for four typical soil [8)]

Table 2-6 Soil laboratory test

		Stability Analysis	Consolidation Analysis	Seismic Analysis	Japanese Standard	International Standard
Physical properties	Specific Gravity Test	++	++	++	JGS0111-2000 JIS A-1202	ASTM D854
	Water Content Test	++	++	++	JGS0121-2000 JIS A-1203	ASTM D2216
	Atterberg Limit Test (Liquid Limit & Plastic Limit)	++	++	++	JGS0141-2000 JIS A-1205	ASTM D4318
	Grain Size Distribution Test	++	++	++	JG0131-2000 JIS A-1204	ASTM D422 ASTM D6913
	Natural Density Test	++	++	++	JGS0191-2000 JIS A-1225	ASTM D7263
	Minimum Density and Maximum Density Test	-	-	+	JGS0161-2000 JIS A-1224	ASTM D7382
Mechanical properties	Unconfined Compression Test	++	-	-	JGS0511-2000 JIS A-1216	ASTM D2166
	Unconsolidated-undrained (UU) triaxial compression Test	+	-	-	JGS0521-2000	ASTM D2850
	Consolidated-undrained (CU) Triaxial Compression Test	+	-	+	JGS0522-2000	ASTM D4767
	Consolidated-undrained ($\overline{CU}$) Triaxial Compression Test	+	-	+	JGS0523-2000	
	Unconsolidated-drained (CD) Triaxial Compression Test		-	+	JGS0524-2000	ASTM D7181
Consolidation properties	Consolidation test (Loading Control)	-	++	-	JGS0411-2000 JIS A-1217	ASTM D2435
	Consolidation Test (Strain Control)	-	+	-	JGS0412-2000 JIS A-1227	
Dynamic properties	Cyclic Undrained Triaxial Test (liquefaction test)	-	-	+	JGS0541-2000	
	Cyclic Triaxial Test (to obtain deformation properties of ground material)	-	-	+	JGS0542-2000	ASTM D5311
	Cyclic Torsional Direct Shear Test (with hollow cylindrical specimen to obtain soil deformation properties)	-	-	+	JGS0543-2000	

**++: Compulsory; +: As required*

2-3-2 Shear Strength

In case a slip failure is expected at a soft ground site, it is necessary to determine the shear strength to perform stability analysis. In case that quick banking embankment is proposed, the shear strengths of the ground are determined by the direct shear test, the unconfined compression test, and the unconsolidated-undrained triaxial compression test. In case that increment of soil strength can be anticipated in slow banking or layer by layer banking, the shear strength of the ground can be determined by the consolidated-undrained triaxial compression test.

The sear strength of the soil does not always stay the same even in a single soil layer. It is necessary to make a comprehensive study of many shear strength values and other soil test results to determine appropriate design values. It is recommendable to conduct shear strength tests on two to three samples, and preferably four or more samples, in a single soil layer. Shear strength tends to be anisotropic (horizontal strength not the same as vertical) as a result of the way a soil mass develops from sedimentation. In view of this observation, current practice suggests that one look at the location of the likely failure mode (shear, compression) and perform a laboratory test consistent with the general orientation of the failure zone as illustrated in Figure 2-25.

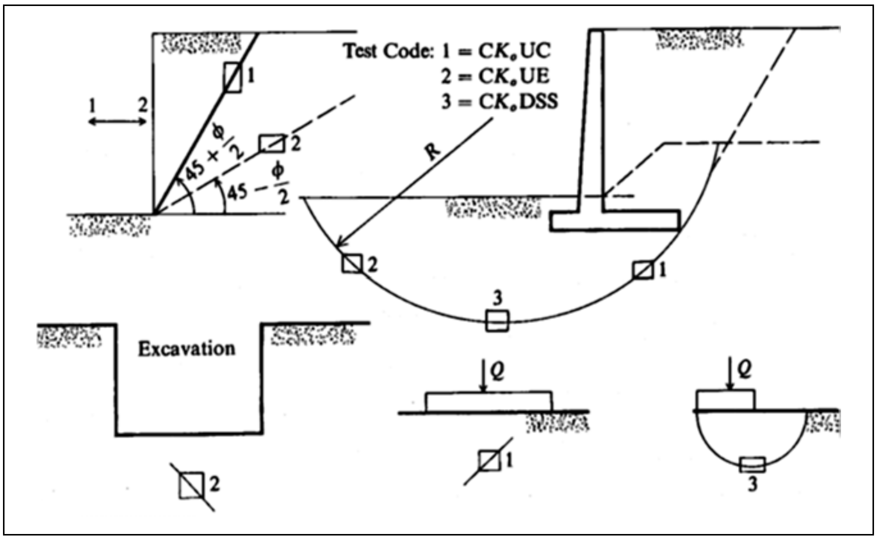


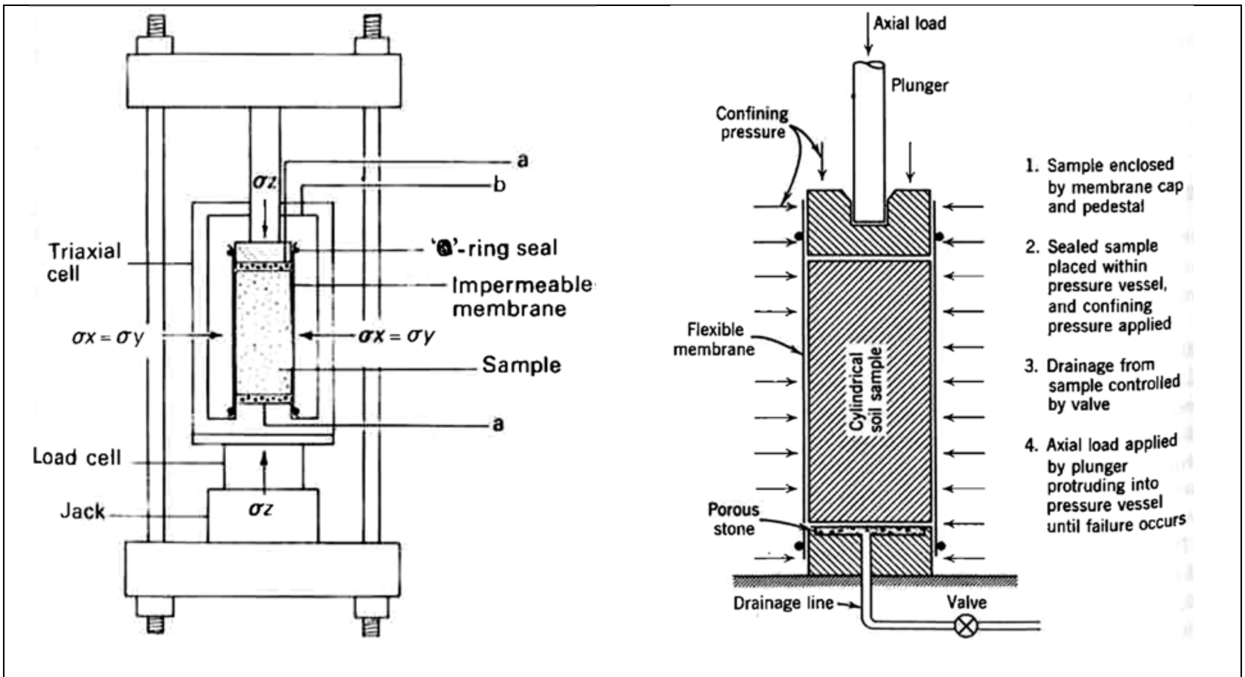
Figure 2-25 Strength tests corresponding to field shear [1)]

Unconfined compression tests to obtain a compressive strength, always termed *qu*, can be performed using almost any type of compression-loading device on cohesive samples. From the peak strength values, a Mohr's circle may be drawn to obtain the untrained shear strength (*su*) as below.

$$su = qu / 2 = cohesion$$

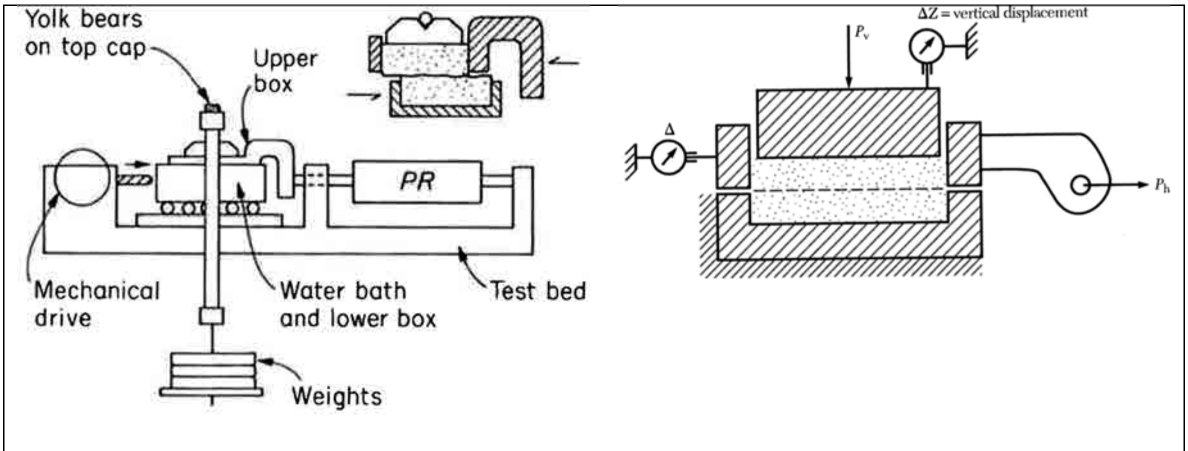
In principle, a triaxial compression test is conducted for at least three specimens with changing pressures to determine a set of shear strength, cohesion and angle of internal friction, (c_u, ϕ_u), (c_{cu}, ϕ_{cu}), or (c', ϕ'). For the pressure (the confining load) used in a triaxial compression test, a number of different pressure values are set between the overburden pressure before banking and the pressure resulting from the addition of overburden pressure up to the maximum embankment load (embankment height × unit volumetric weight of banking material).

The direct shear test can be used to determine the shear strength of soils, but it is not used in preference to the triaxial test because of difficulties in controlling drainage conditions, and the fact that the failure plane is predetermined by the apparatus. However, the reversing shear box provides a useful means of obtaining the residual or long-term shear strength used in calculation the stability of earth slopes where failure may take place on an ancient slip surface.



Rudiments of tri-axial cell [5]   Essential feature of tri-axial test [2]

Figure 2-26 Tri-axial compression apparatus



Rudiments of direct shear apparatus   Cross sectional diagram of direct shear box

Figure 2-27 Direct shear test apparatus [2]

Figure 2-28 Direct shear apparatus in soil research laboratory, Public Works

2-3-3 Soil Properties for Consolidation

Consolidation tests are conducted to analyze the amount of settlement resulting from construction of an embankment or the rate of this settlement. The consolidation properties normally required are the coefficient of compressibility which is the change in unit volume and in pressure, and the coefficient of consolidation which is proportional to the ratio of the coefficients of permeability and compressibility. In an oedometer which is the simplest consolidation test apparatus, a sample is compressed with a known load and the resulting changes in its volume with respect to time are measured. In this equipment a sample is axially loaded in one direction only, i.e. vertical. The maximum consolidation pressure is set so that it exceeds the overburden pressure before banking plus the pressure of the embankment load (embankment height × unit volumetric weight of banking material). Since consolidation properties are not always uniform in a single layer, a consolidation tests shall be conducted for as many points as possible so that the design values can be rationally determined. As a consolidation test is one of the most important tests for soft soil investigation, at least one specimen is taken from a single sampling tube and subjected to a consolidation test.

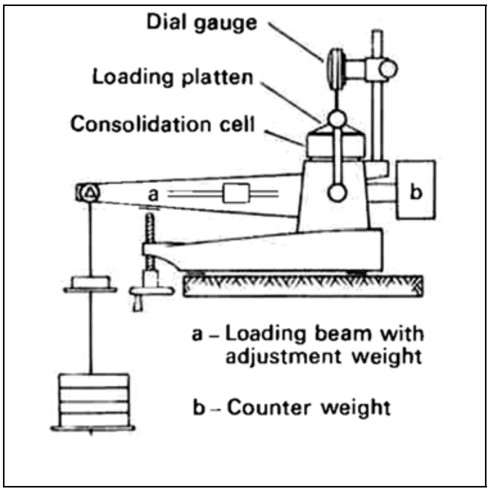


Figure 2-29 Oedometer [2)]

Figure 2-30 Oedometers in Bridge Research Laboratory, Highway Department, MOC

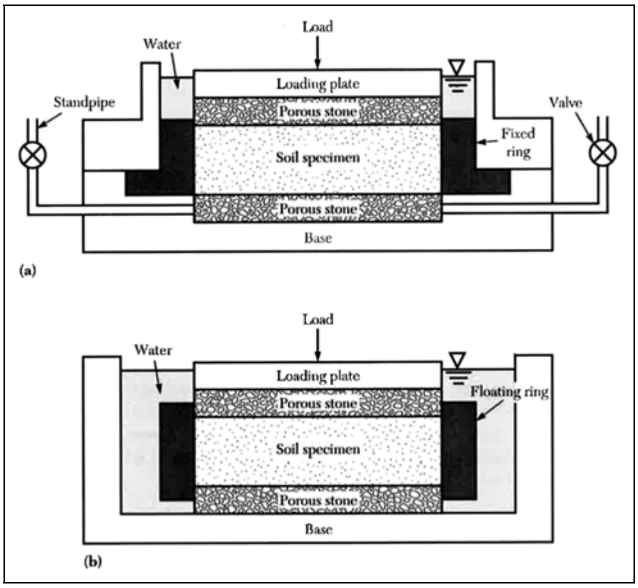


Figure 2-31 Consolidation cell [2)]

a: Fixed-ring container, b: Floating-ring container

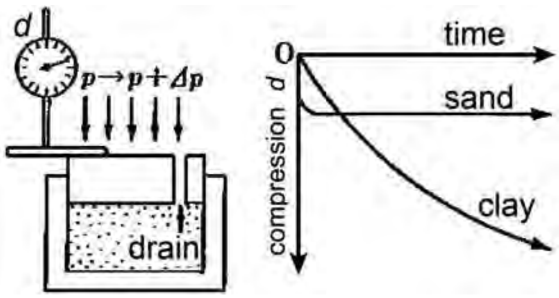


Figure 2-32 Change in compression with time [6)]

2-3-4 Soil Properties for Seismic Analysis

Liquefaction evaluation, stability analysis, study of residual deformation in an earthquake, etc., are conducted in a study of earthquake resistance. Many of the seismic properties necessary for a study of earthquake resistance may be determined from the N-values obtained from the standard penetration test, the results of physical tests of samples taken with samplers for the standard penetration test, and constant strength properties. However, in some cases, additional dynamic soil tests may be necessary depending on the ground conditions, study methods, and the target precision.

Liquefaction evaluation is usually carried out based on the liquefaction resistance ratio F_L (= R/L) defined by the ratio of the dynamic shear strength ratio R to the seismic shear stress ratio L (see "3-8 Verification of Stability against the Actions of Seismic Motion"). In cae a dynamic shear strength ratio is estimated from a standard penetration test, N-values as well as the results of physical tests such as the fine-grain content ratio are required. Therefore, for a soil layer assumed to be vulnerable to liquefaction, it is recommendable to perform physical tests with specimens taken using standard penetration tests carried out at as many depths as possible within the reachable range. For the design of an important, large-scale earthwork structure, the dynamic shear strength ratio can be calculated by applying the results of a cyclic triaxial test (used to determine the deformation properties of ground data), and a cyclic torsion shear test with a hollow-cylinder test specimen (used to determine the deformation properties of soils) in the ground response analysis. In particular, for sandy soil with less fine particle content, data obtained from tube sampling are generally subject to disturbance caused during sampling and there is a risk that they can not be used for an appropriate evaluation of the dynamic shear strength ratio. Therefore, for testing on sandy soil with less fine particle content, it is necessary to select a sampling method that involves less disturbance, such as frozen sampling.

2-3-5 Soil Tests for Embankment Material

For an embankment material study, the following tests are necessary to

a. determine physical properties, such as
   - Specific gravity of soil solids
   - Particle size distribution,

b. determine compaction properties, such as
   - CBR on compacted soil
   - Cone index of compacted soil

Tri-axial compression test on compacted soil is also employed to obtain the shear strength of the embankment.

2-3-6 Verification of Test Results

Prior to setting the soil properties, it is important to evaluate the appropriateness of the soil test results, rather than simply using the results as they are. The test results are evaluated with regard to whether or not correct test values were obtained by considering the points noted below. These points, however, are applicable to soils with the same properties and not to soils having different properties. Whether or not the soil properties are different is evaluated using the soil particle density, particle size composition, consistency, etc.

a. The value of wet density ordinarily decreases with a rise in the water content ratio, so if there is a deviation from this tendency, it is necessary to verify the causes. For 100% saturated cohesive soil below the groundwater level, it is generally to consider that the correct density can be obtained. For non-saturated soil although the soil is below the groundwater level, however, it is necessary to check the water content ratio, wet density, soil particle density, and other values.

b. If a specimen is disturbed during sampling or testing, its yield strain tend to be enlarged in an unconfined compression test. In addition, it is generally understood that the unconfined compressive strength increases as the depth increases. If a specimen has sandy content, it is more strongly affected by stress release, and ultimately the shear strength of the ground can be underestimated. In such cases, it is necessary to also use an unconsolidated-undrained triaxial compression test or to correct the unconfined compressive strength with clay content or a plasticity index to re-evaluate the strength.

c. In ordinary situations, compressive yield stress increases as the depth increases. If the results do not much this tendency, it is necessary to check the physical properties of the specimens and the sedimentary environment of the soil layer samples.

d. It is common to see a high correlation between compression index and water content.

e. Although the consolidation coefficient is large in the stress range of over-consolidation and small in the stress range of normal consolidation, since there is usually no major change in each range, the compression theory is often applied with the consolidation coefficient can be regarded as constant. In practice, because of the small compressibility of over-consolidated soil, the calculation of consolidation covering from over-consolidation to normal consolidation is conducted only with the consolidation coefficient in the range of normal consolidation. Caution is necessary if the average over both ranges is used, because it could result in estimation of a faster consolidation rate.

2-4 Soil Report

The results of ground exploration, including sounding test results, field test results, boring logs, and soil test results, are summarized as “soil property logs” for each investigation point. These soil property logs are then rearranged to prepare “soil profiles,” which are compiled together with plan views that clearly indicate the investigation points. In this general data compilation process, the results are reorganized so that they can be effectively used to study settlement, stability, and liquefaction. For example, if the soil layers are divided into too many segments, the study could become very complicated. On the other hand, if soil layers are only roughly classified, soil test results that indicate soft soil layers and sandy layers that cause problems in the study of settlement, stability, or liquefaction could be missed. Therefore, appropriate handling is required. Investigation results are summarized according to the procedure shown in Figure 3-10.

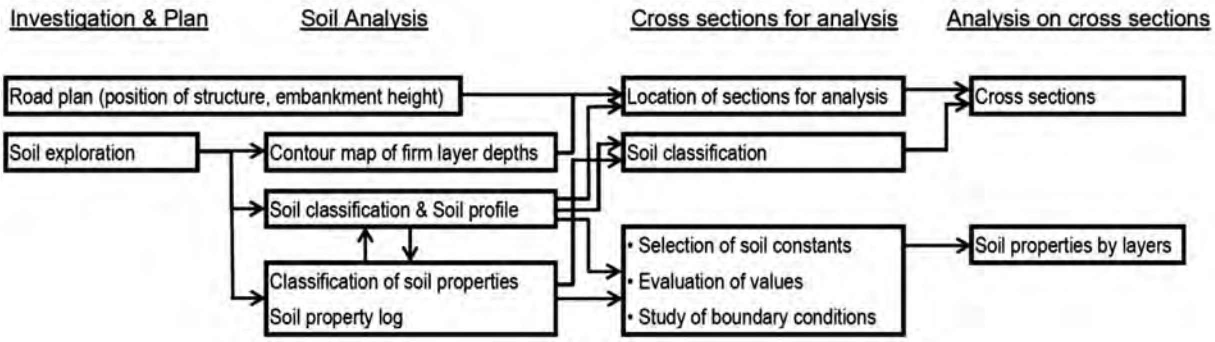


Figure 2-33 Procedure for summarizing investigation results and determining soil properties

2-4-1 Soil Property Log

Soil property log that summarize the results of soundings (standard penetration tests, etc.), in situ tests (vane tests, etc.), and soil laboratory tests in the depth direction are prepared together with boring logs for each boring point for the soil classification and design soil properties (which are design conditions). PS logging result is sometimes organized in the depth direction like soil property log. An example of a soil property log is shown in Figure 2-34. The soil property log as example in Figure 2-34 shows that the test results plotted on the horizontal axes sets for each of the items on a scale of about 1/100 on the vertical axis.

- Soil name
- Particle size composition: The composition of clay content, silt content, sand content, and gravel content is shown.
- Consistency index (liquid limit w_L, plasticity limit w_p), natural water content (w_n)
- Bulk density (ρ_t), density of soil particles (ρ_s), void ratio (e)
- Standard penetration test results (N-value), unconfined compressive strength (q_u), unconfined compression failure strain (ϵ_f)
- Consolidation yield stress (p_c), effective overburden pressure (p_0), compression index (C_c)

(Since the compressibility coefficient (c_v) and volume compressibility coefficient (m_v) vary depending on the consolidation pressure, they are usually not included in the list.)

- Direct shear test results or triaxial compression test results $(c_u,\ \phi_u)$, $(c_{cu},\ \phi_{cu})$, or $(c',\ \phi')$ may be described if necessary.
- Modulus of elasticity (initial tangent modulus E_0, secant modulus E_{50})

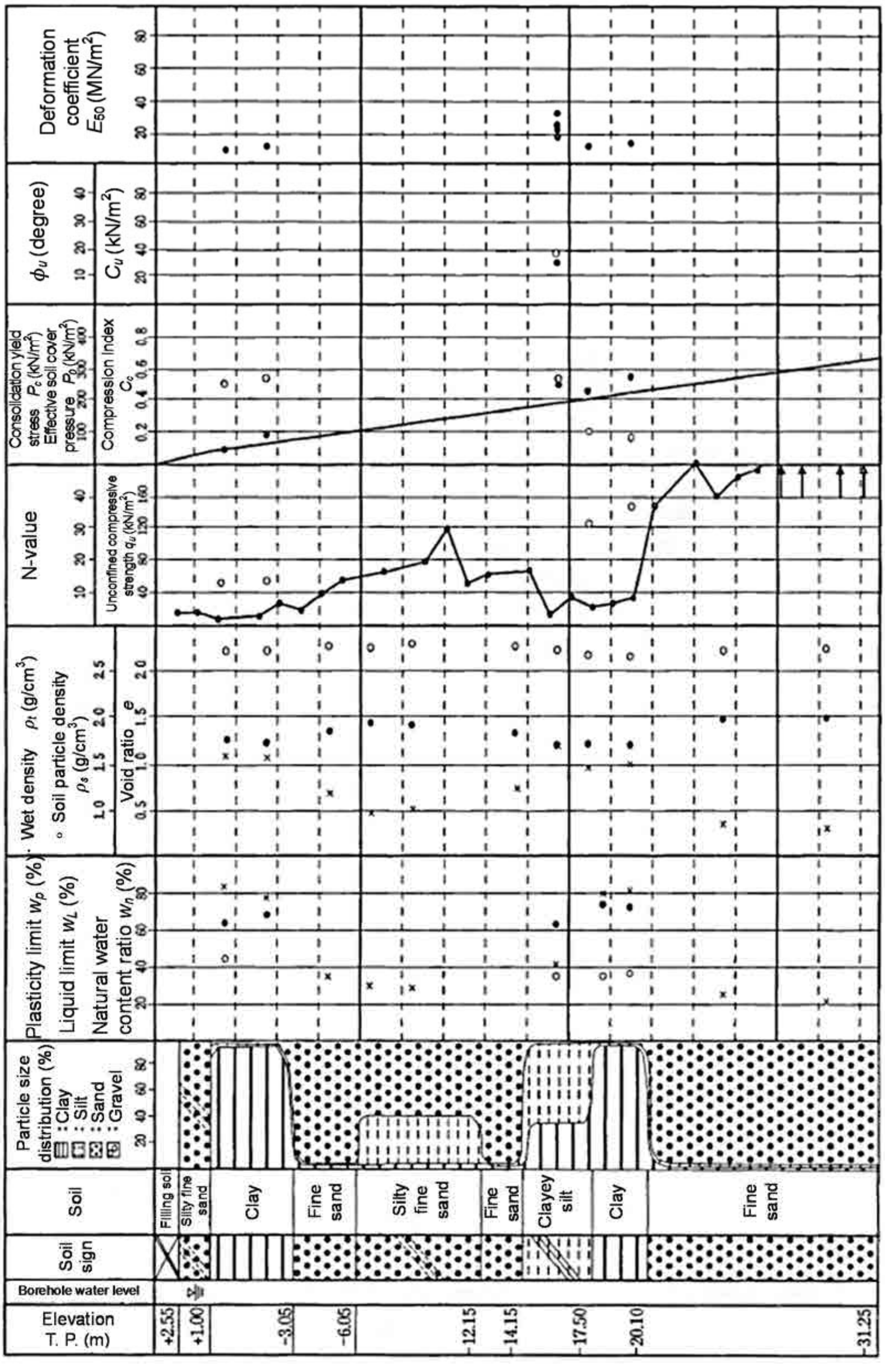


Figure 2-34 Example soil property log

Figures 2-35 is examples of soil property log. The purposes of presenting the profiles is to ;

- Indicate how geological history influences soil properties.
- Give typical values of soil properties.
- Show dramatically the large variability in soil behavior with depth.
- Illustrate how engineers have presented subsoil data.

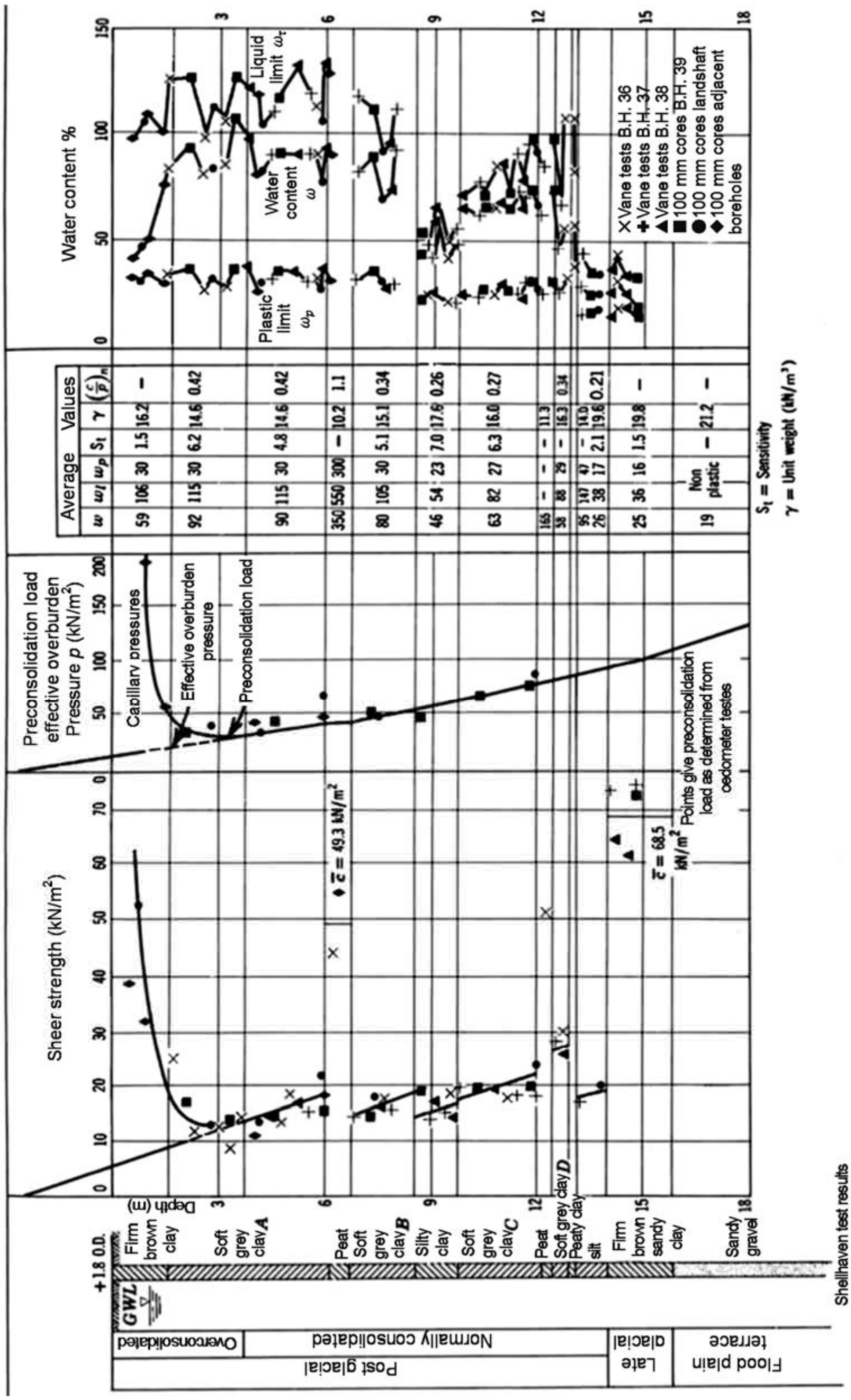


Figure 2-35 Example of soil property log [1)]

2-4-2 Soil Profile

The first step of establishing conditions for study is to clarify the scale of soft ground and the continuity of soil composition. For this purpose, a soil profile log is prepared using the sounding and boring results. A soil profile is used to clarify the soil composition status of the ground. Soil layers are classified not only based on the boring and sounding results, but also by considering the soil test results and sedimentary environment shown in the soil property log, and all investigations and tests are arranged to complete a soil profile. An example of a soil profile prepared using this procedure is shown in Figure 2-36.

Since boring or sounding is generally conducted along the centerline of the road, a soil profile is first prepared in the longitudinal direction of the road. A boring log including the standard penetration test results and the results of sounding tests for each investigation point is entered on a longitudinal section prepared from the location investigation of a route at a scale of 1/100 or 1/200 in the vertical direction so as to prepare a soil profile. It is important to keep in mind the following points in preparing a soil profile. If a soil layer found at one borehole is not found at the other adjoining borehole (such as when an intermediate sandy layer exists in a lenticular pattern), the spread of the soil layer can be evaluated by sounding tests conducted between the two boreholes. If necessary, additional sounding or boring investigations are carried out.

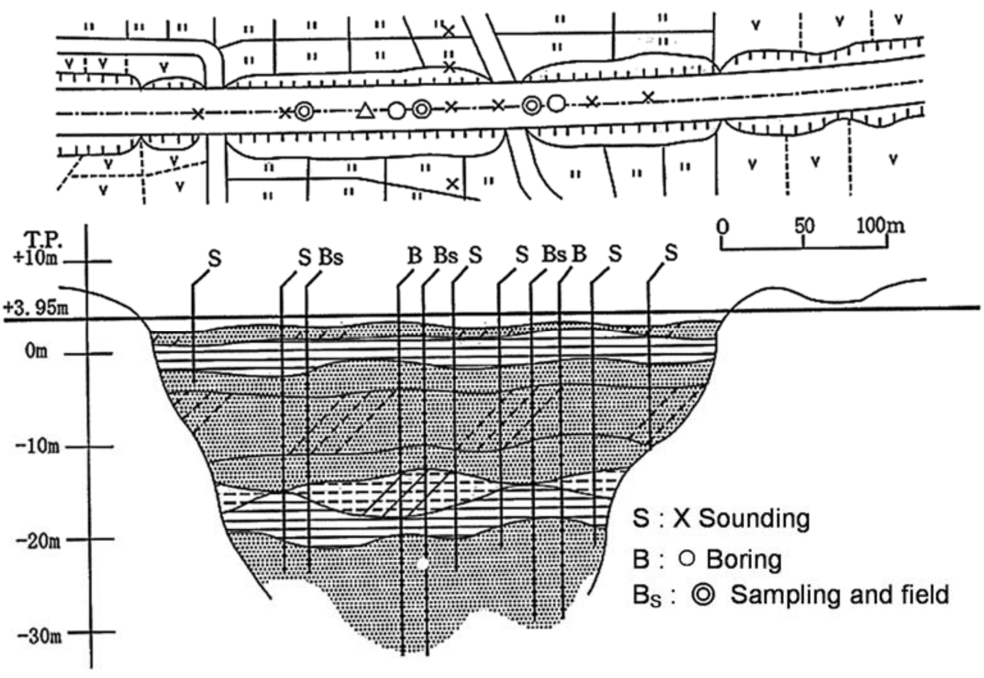


Figure 2-36 Example of soil profile

It is important for a soil profile to show the position of the bearing layer (foundation) under the soft soil layer and its geology, particularly whether it is a permeable layer or a low-permeability layer, and the presence of an intermediate sandy layer in the soft layer. It is sometimes necessary to prepare a soil profile in the transverse direction in addition to the longitudinal direction of the road.

When a road crosses a narrow valley, it can be suspected that the thickness of a soft layer or the soil composition drastically changes in the transverse direction. In addition to boring at the road center, sounding tests are conducted at least near the slope toe on both sides of the embankment so that a profile in the transverse direction can be prepared. Furthermore, the same consideration is required when an embankment is built over a wide area for an interchange or an approach road.

2-4-3 Cross Section

Prior to design calculation, a cross-section to study in the transverse direction is prepared based on the soil profile of the ground in the longitudinal direction as shown in Figure 2-37. Factors to be clarified for the soil classification include: (a) the thickness of a soft soil layer (the location of the foundation, which is a strong layer under the soft layer); (b) the boundary conditions of the lower part of the soft layer (whether the foundation is a low-permeability layer or a permeable layer); (c) the presence or absence of an intermediate sand layer that serves as a draining layer; and (d) soil classification of the soft layer (to identify the layers among the soft layers that have different properties). These items should be evaluated by comprehensive study of various test results such as boring, sounding, soil testing, or field reconnaissance, as well as geological study results such as classification of alluvial layers and diluvial layers.

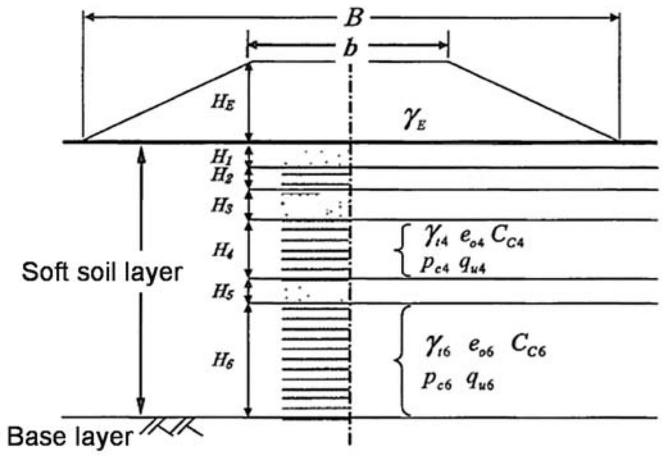


Figure 2-37 Soil classification

The basic points for soil classification include the following.

a. Even if a soft soil layer is thick and its layer composition is complicated, it is recommendable to limit the number of layers to a maximum of 5 to 6. For integration, it is particularly recommendable to choose layers with no continuity or layers that do not function as draining layers and that are similar in soil properties.

b. Although thin sand layers with good continuity are ignored for stability and settlement analysis, they should be considered as draining layers in the boundary conditions for consolidation analysis.

c. When the inclination of soil layers or thickness changes in the transverse direction are clarified in soft ground formed in a buried valley, these points should also be considered in the soil classification.

d. Soil section should be conducted based on soil classifications. This rule makes it possible to confirm intermediate permeable layers or permeable layers in the foundation and to differentiate silt, clay, and organic soil in a soft soil layer. If it continues well in the lateral direction, a sandy soil layer can serve as an effective draining layer.

e. The foundation of a soft layer should be evaluated according to sounding results. The soil layer of which N-value is over 4 to 6 for cohesive soil and over 10 to 15 for sand or sandy soil, can be considered as a firm base layer. When a Swedish sounding test is used, the cohesive soil layer with more than 100 half-turns per meter can be considered as a firm base layer. When a Dutch double-tube static cone penetration test is used, the soil layers with over q_c = 1,000 kN/m2 for cohesive soil and over q_c = 4,000–6,000 kN/m2 for sand or sandy soil can be considered as a firm base layer. Since the foundation changes depending on the applied load, the load of the earthwork structure to be constructed is assumed, and the foundation of the soft layer is studied. It is also essential to make sure the foundation has continuity in the depth and horizontal direction.

f. Soil test results should be used to evaluate whether or not there is further subdivision of a soft soil layer that is assumed to be of a single soil classification. In this case, the following require attention: natural water content, wet density, unconfined compressive strength, consolidation yield stress, and the consolidation index. When soils have clearly different values for these factors even though they are of the same soil type, they should be taken as different layers. In this evaluation, sounding results may be used for reference.

Layer classification is determined based on a comprehensive evaluation for each of the above items. A different set of layer classification items may be used depending on the purpose of the calculation.

1) Joseph E Bowls (1997); Foundation Analysis and Design, McGraw-Hill
2) A.M. Al-Khafaji et al. (1992); Geotechnical Engineering and Soil Testing
3) MJ Tomlinson et al. (1995); Foundation Design and Construction, Longman Scientific and Technical
4) Public Works Research Institute Japan (2004) ; Manual for Highway Earthworks in Japan (English)
5) Dimitri P. Krynine (1957); Principle of Engineering Geology and Geotechnics, McGraw-Hill
6) Japan Society of Civil Engineering (1989); Handbook of Civil Engineering (Japanese)
7) T. William Lambe (1979); Soil Mechanics, SI version, John Wiley & Sons, Inc.
8) F.G.H. Blyth et al.(1988); A Geology for Engineers, ELBS
9) (Gouda Geo Equipment B.V. http://www.gouda-geo.com/products/cpt-equipment)
10) (OYO Corporation, www.oyo.co.jp/english/products/products-list/)

* TCP : Technical Cooperation Project (JICA Expert Team)

Chapter 3

Design of Earthwork Structures on Soft Ground

3-1 Basic Matters on Design

Since characteristics of the ground are generally complicated, it is often difficult to accurately understand the behavior of the ground or soft ground treatment in the investigation and design stage, and to precisely predict the behavior of the earthwork structure during or after construction or its impact on the surroundings. Therefore, it is important to investigate past construction works conducted at locations with similar soil properties or past cases of accidents at these locations, and to make decisions from a comprehensive viewpoint in designing earthwork structures on soft ground or treatment work. In addition, experimental construction in important and instrumentation is important to monitor the construction works and feedback information on design and construction methods.

When developing the design of an earthwork structure on soft ground, it is necessary to examine the results of soil investigation in detail, and appropriately predict the behavior of the earthwork structure and the ground. However, because the variations in layer thicknesses or soil properties of soft ground are usually complicated, it is difficult to grasp them in advance. Although the analysis techniques for ground behavior are making progress, it is often difficult to accurately predict the stability of an earthwork structure on soft ground with studies only. Therefore, decisions are not made simply on the basis of study results. When designing a structure, a comprehensive decision must be made based on the records of works in the vicinity or similar structures or similar types of ground.

In the design of an earthwork structure and its secondary structures, it is important to take these prediction uncertainties into account and review a structure capable of coping with settlement. For example, extra allowances are included in the design of width of an embankment or the size of a culvert to cope with reduction in the width of the embankment, insufficiency of the inner section of the culvert, etc., caused by residual settlement. When designing structures like culverts or secondary structures, it is important to study the selection of a type of structure or foundation that can follow the movement of settlement, as well as the joint intervals to be set and the joint structures to be adopted. The details of the structures are described in “4-2-5 Applicability of Treatment Methods at Peculiar Places” for details.

It is also important to carry out experimental construction in advance in order to check or revise the soil parameters used in the design, and to confirm the applicability of construction methods or new methods and technologies. Furthermore, it is recommendable to monitor ground movement during the works to try to obtain more accurate ground behavior and feedback information to revise design and construction methods as required prior to construction. Specifically, the workflow for checking the stability of an earthwork structure involves considering constraints that apply to construction of the earthwork structure, determining check items and their tolerances, checking the stability of the soft ground and the earthwork structure against each action based on detailed ground information, and final determination of the structure. Constraints applied here include design conditions that are hard to change (e.g., road and structure conditions, ground conditions, and secondary structures) and conditions that can be changed or revised during the process of studying the work plan (e.g., construction conditions including the earth volume, construction period and construction cost, and maintenance conditions).

In the stability analysis of an earthwork structure on soft ground against normal actions, the basic procedure of the analysis is to ensure that the earthwork structure will be stable against loads during the construction of the earthwork structure and the road service period (including dead loads and burden loads), and to make sure there will be no settlement with a potential negative impact on the functions of the earthwork structure (i.e., road surface trafficability) and no damaging deformation (settlement, uplift, etc.) in the surrounding facilities or ground. These check items, however, may be abbreviated depending on the characteristics of the earthwork structure or the status of the surrounding facilities. For example, checking of deformation of the surrounding ground may be abbreviated if the amount of settlement is small, if sufficient stability can be maintained, and if there are no important facilities in the surrounding area.

3-2 Forces act on Ground and Earthworks

3-2-1 Considerable Actions

There are some actions that have to be considered in designing an earthwork structure on soft ground or treatment work shall be selected according to various conditions including the location of the earthwork structure.

(1) Regular Actions

Regular actions are actions always act on an earthwork structure and soft ground treatment work as follows.
- dead loads,
- burden loads,
- water pressure (influence of rising groundwater due to rainfall),
- water pressure (infiltration water from rivers or ponds),

(2) Rainfall actions

Rainfall action is considered as appropriate depending on the followings.
- type of earthwork structure
- type of soft ground treatment work
- construction conditions
- local rainfall characteristics
- characteristics of the location of the earthwork structure
- degree of importance of the road
- simultaneous use of advance traffic regulation
- other relevant factors

(3) Seismic motion actions

The assumed actions by seismic motion are classified into Level 1 and Level 2. Level 1 is seismic motion that has a high possibility during the service period of the road. Level 2 is seismic motion that has a lower possibility, but that has a large intensity. Furthermore, Level 2 can be classified into two types that need to be considered: Type 1 is assumed to be produced in a large-scale earthquake at a plate boundary; and Type 2 is assumed to be produced in an inland earthquake. If the seismic motion can be estimated based on the seismic motion for the proposed site can be set.
- past earthquakes at the proposed site and the surroundings,
- active faults,
- earthquakes occurring at plate boundaries,
- underground structure,
- ground conditions of the subsurface layers
- records of the past major earthquakes.

(4) Other actions

Action by other factors includes environmental actions like frozen soil due to low temperatures, salt damage impact, and corrosion in acid soils. These actions are considered as appropriate depending on the type of earthwork structure or soft ground treatment work or their construction conditions.

3-2-2 Considerable Loads

(1) Types of Load

Generally, for the designing of an earthwork structure or treatment works on soft ground, design loads are appropriately considered from the followings depending on the structure's construction location and structure.

- Dead loads
- Burden pressure
- Earth pressure
- Water pressure and buoyancy
- Impacts of earthquakes

These loads are the main loads to be considered for analysis of stability in designing an earthwork structure on soft ground or treatment work. They are appropriately selected depending on various conditions of the construction site location or the structure type, and it is not necessary to use them all. The load of construction machines on the soft ground is explained in "4-2-3(4) 3) Trafficability of construction machines". The loads shall be considered as acting to the greatest disadvantage of the earthwork structure within the expected range.

(2) Combinations of loads

Appropriate load combinations need to be set by taking into consideration the conditions that produce the greatest disadvantage among the combinations of loads most likely to act at the same time. Examples of typical load combinations for earthwork structures are shown in Table 3-1.

Table 3-1 Load of embankment

Actions	Loading condition	Load
Regular actions	During construction	Dead loads (+ burden loads)
	After construction	Dead loads (+ burden loads)
Actions of ground motion	Seismic motion Level 1	Dead loads + seismic impact
	Seismic motion Level 2	Dead loads + seismic impact

( ) Loads in parentheses are considered to be required according to the construction conditions or the effect on the stability of the earthwork structure.

(3) Dead Load

Dead load of embankment

The dead load of an embankment, including the pavement part, can be determined by multiplying the wet unit weight of the banking material γ_t by the cubic volume of the embankment (including the pavement part). Although the unit weight of the banking material should be calculated by using the actual banking material used in the work with consideration of the construction conditions, it is often not possible to calculate the unit weight of a banking material with the soil test in the preliminary study stage. In such cases, it is acceptable to assume the unit weight as per Table 3-3

Table 3-3 Unit weight of soils of embankment and natural ground

Ground	Soil	Unit weight γ_t (kN/m3) Loose	Dense
Natural ground	Sand and sandy gravel	18	20
	Sandy soil	17	19
	Cohesive soil	14	18
Embankment	Sand and sandy gravel	20	
	Sandy soil	19	
	Cohesive soil with w_L < 50%	18	
	Volcanic cohesive soil	15	

Dead loads of structures such as retaining walls or culverts

The values shown in Table 3-4 can be used as the unit weight of a material r used in calculating the dead loads of retaining walls, culverts, and other structures in an initial analysis.

Table 3-4 Unit weight of material of structures

Material	γ (kN/m3)
Concrete	23.0
Reinforced concrete	24.5

(4) Burden Load

Burden loads are appropriately determined by considering the type of structure, vehicle traffic conditions, or status of construction works. In considering traffic loads or work loads when studying the stability of embankments or retaining walls, 10 kN/m^2 can be used as the burden loads.

(5) Earth Pressure

Normal earth pressure is appropriately considered and calculated according to the ground conditions, earthwork structure conditions, and treatment methods.

(6) Water Pressure and Buoyancy

Water pressure

Water pressure is considered appropriately with the ground conditions and variation of water level due to rainfall. Water pressure is considered when an earthwork structure is constructed on a level lower than the groundwater level.

Buoyancy

Buoyancy is considered appropriately based on variation of pore water pressure and water level. Buoyancy is considered when an earthwork structure is constructed on a level lower than the groundwater level due to the influence of rainfall. Buoyancy is taken to act upward and applied to an earthwork structure to its greatest disadvantage.

3-2-3 Seismic Motion

The following impacts of earthquakes are appropriately considered according to the type of structure.

- Inertial force attributable to the weight of a structure
- Earth pressure during an earthquake
- Displacement or deformation of the surrounding ground during an earthquake
- Dynamic water pressure during an earthquake
- Impact of liquefaction

The kinds of earthquake impacts that should be considered in the design of an earthwork structure on soft ground are shown here. Based on the relevant guidelines for earthwork structures ("Road Earthworks: Embankment Work Guidelines", "Road Earthworks: Retaining Wall Work Guidelines", and "Road Earthworks: Culvert Work Guidelines"), these earthquake impacts and their combinations are appropriately set according to the ground conditions, earthwork structure conditions, and treatment methods.

Inertial force

Inertial force is considered when it is estimated to have a large impact during an earthquake. Since deformation or damage caused to an earthwork structure on soft ground or treatment work due to an earthquake is usually dominated by the impact of earthquake motion in the horizontal direction, it is acceptable not to consider the influence of inertial force in the vertical direction. When checking is conducted using a static checking method, the inertial force calculated by multiplying the weight of the structure by the design horizontal seismic intensity in the horizontal direction is taken to act on the structure. When study is conducted using a dynamic analytical method, the time history of input earthquake motion is necessary. In this case, acceleration waveforms with spectral characteristics similar to the target acceleration response spectra should be used as suggested by the "Specifications for Highway Bridges, Part V: Seismic Design (March 2002)". In the design, when the location of earthquake motion input is set as the base ground surface, the design of earthquake motion waveforms is determined with appropriate consideration of the impact of the ground.

Impact of liquefaction

If liquefaction potential layer is deep and thin, it might have little impact on the functions and safety of the earthwork structure. Even if there is a potential layer for liquefaction, the impact of liquefaction can be neglected if the impact of liquefaction on the structure is expected small, it is not necessary to consider the impact of liquefaction.

3-3 Soil Properties

In the design of an embankment on soft ground, stability calculation and settlement calculation are mainly carried out, and in some cases, liquefaction analysis is also performed. The ground conditions for such design calculations and design are determined from the soil investigation results. Here, the term “ground conditions” includes the locations and thicknesses of cohesive soil layers or organic soil layers, which are generally taken to be:

- soft soil layers such as clay and organic soil,
- loose sandy soil layers with liquefaction potential,
- permeable soil layers,
- hard clayey layers.

Since these soils are not uniform in quality and their properties change in both the horizontal and vertical directions, it is difficult to have a full understanding of the ground conditions with soil investigation. Even if detailed conditions are obtained, it is impossible to input excessively complicated soil information into a calculation. In setting the ground conditions, the overall status of the soft ground is assumed based on the soil investigation results at individual points, the appropriateness of the obtained soil test results is evaluated, appropriate values are selected, and the condition of the ground to be used for the design and the relevant soil properties are determined. In general, the procedure for setting ground conditions for design based on the soil investigation results is as follows.

a. If any abnormal values are found comparing other test results, cause of abnormal values should be found and correction or selection of test results should be done. Classification of the soft soil layers, especially the boundary of permeable layer and impermeable layer should be cleared.

b. Cross-sections that need calculation are selected, and the soil properties to be used are set.

For the values used for stability calculation or settlement calculation for a section under study, average or representative values from the soil test results are used for each classified soil layer. In this operation, the soil properties used in the study of the soft ground must be determined based on a comprehensive judgment for the soft ground as a whole. If there are a small number of constants among the candidate values that are extremely different from the rest, they are excluded. Here, the term “representative values” refers to soil test results for a portion of a soil layer that can be evaluated to be representative of that layer based on the results of other tests. It is recommendable to prioritize representative values that are selected through the discretion of the engineer rather than adopting the average value through mechanical calculation. The basic principle is to determine a design value through an evaluation of the soft ground as a whole, and not simply from a single boring site. However, when a soil with a certain specific cross-section is especially soft, a special value that particularly matches that cross-section alone is determined in some cases where the actual conditions need to be reflected. When soil properties need to be estimated or revised based on experimental construction results or project data precedents, it is necessary to comprehensively evaluate the obtained ground information or the behavior of the earthwork structure and then determine the soil properties. The following points are considered when determining the design values based on the soil test results.

3-3-1 Soil Properties for Consolidation Analysis

(1) Determination of Properties

The average of each of the constants by layer for peat layers or clay layers, as respectively shown in Figure 3-1, are taken as the representative values.

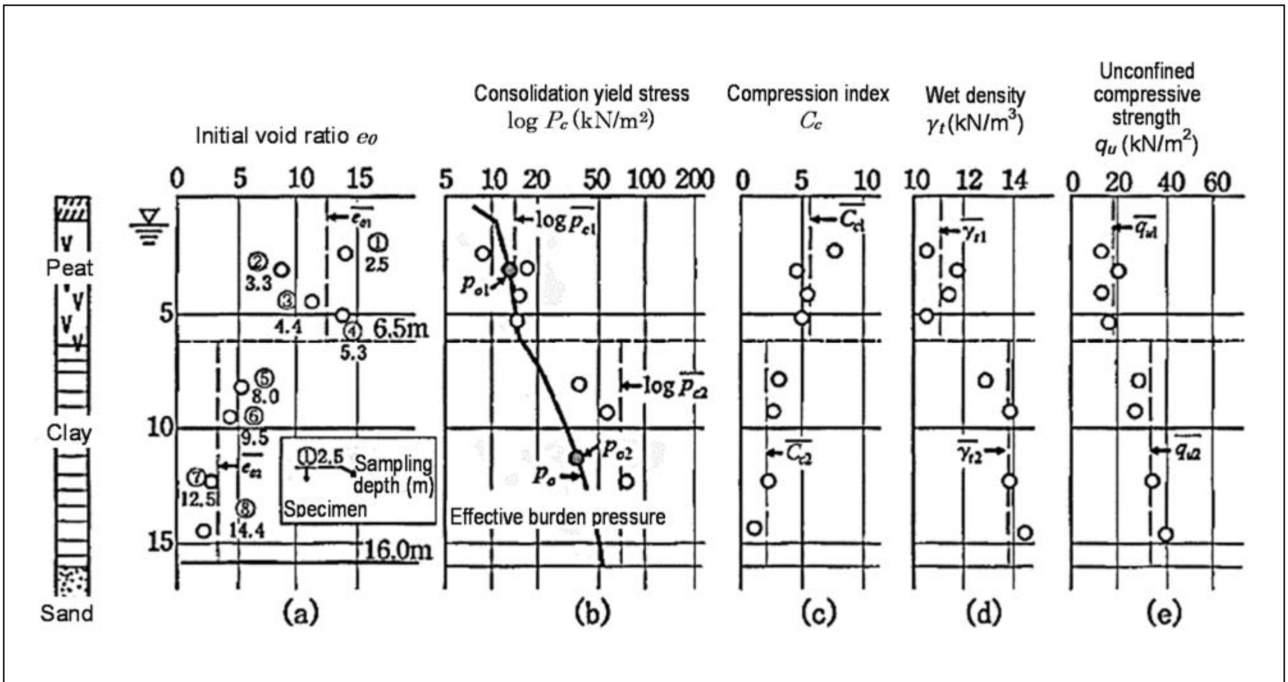


Figure 3-1 Example of soil properties

For the representative value of effective overburden pressure p_0, however, the value at the central depth of each layer is calculated with consideration of the groundwater level. Half the value of the unconfined compressive strength is usually taken as the value of the non-drained shear strength of a soft layer used in stability calculations. Unconfined compressive strength often increases as the depth increases and may greatly change in a single classified layer. In such cases, it is better to take a value that gradually increases with the increase in depth rather than a constant value for each layer. When the value for unconfined compressive strength is small, if it is excluded uncritically, it could lead to an outcome on the dangerous side, and so the significance of the value requires attention. When a loose sandy layer is inter-bedded in a soft layer or a sandy soil layer is very thin, it can be ignored in stability calculations and taken as having the same properties as that of the cohesive soil layer on or beneath it. However, when there is a succession of inter-bedded sandy layers and they are considered to sufficiently function as a draining layer, this is considered in calculations of the rate of settlement caused by consolidation settlement. When a sand layer or sandy soil layer inter-bedded in a soft soil layer is thicker than about 1 m, it is used in stability calculations, and the angle of internal friction ϕ used for it is either determined from the sounding test result or assumed to be about 25° to 30°.

(2) Coefficient of Volume Compressibility mv

The average coefficient of volume compressibility $\overline{m}_v$ of a soft layer may be used to calculate the rough value of ground settlement. Figure 3-2 is the result of calculation of the relationship between the average water content ratio $\overline{w}_n$ and the average coefficient of volume compressibility $\overline{m}_v$ from the data for past expressway and national highway embankment works.

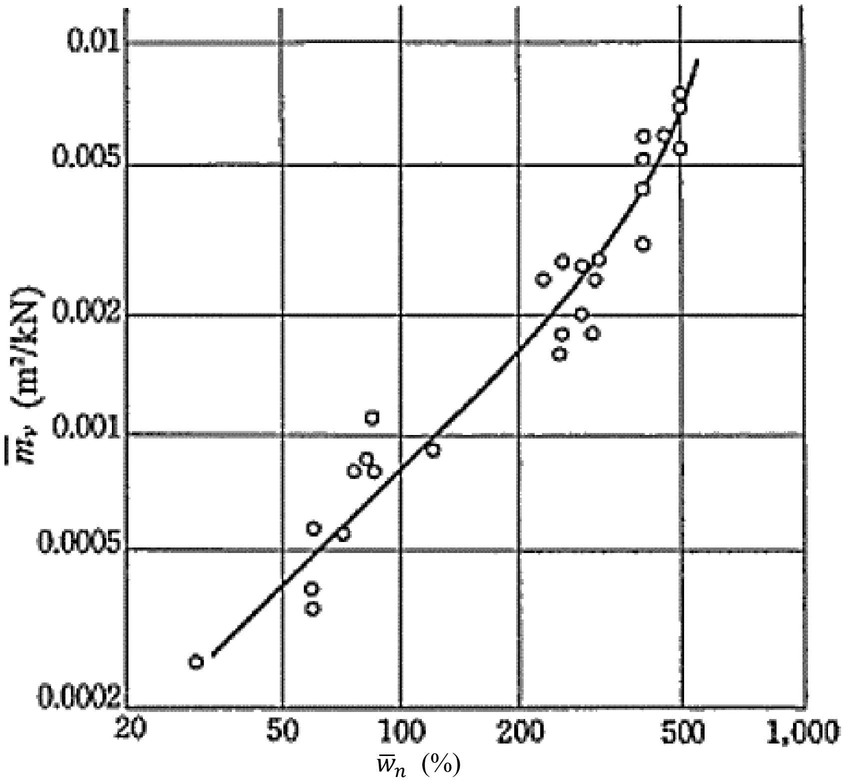


Figure 3-2 Relationship between average water content ratio $\overline{w}_n$ and average coefficient of volume compressibility $\overline{m}_v$

(3) $\boldsymbol{e}$- $\mathbf{log}\,\boldsymbol{p}$ Curve

Water content ratio and e- $\log p$ curve

Figure 3-3 shows the result of statistical processing of the e- $\log p$ curve using the range of natural water content ratio w_u as a parameter for many soft ground sites. Thus, when no e- $\log p$ curve is obtained from testing, the rough settlement amount may be estimated from Figure 3-3 using the known range of the water content ratio of the ground.

For a certain type of ground, like peat ground, it is divided into layers with very different water content ratios, the amount of settlement for each layer should be calculated from the average water content ratio of each layer.

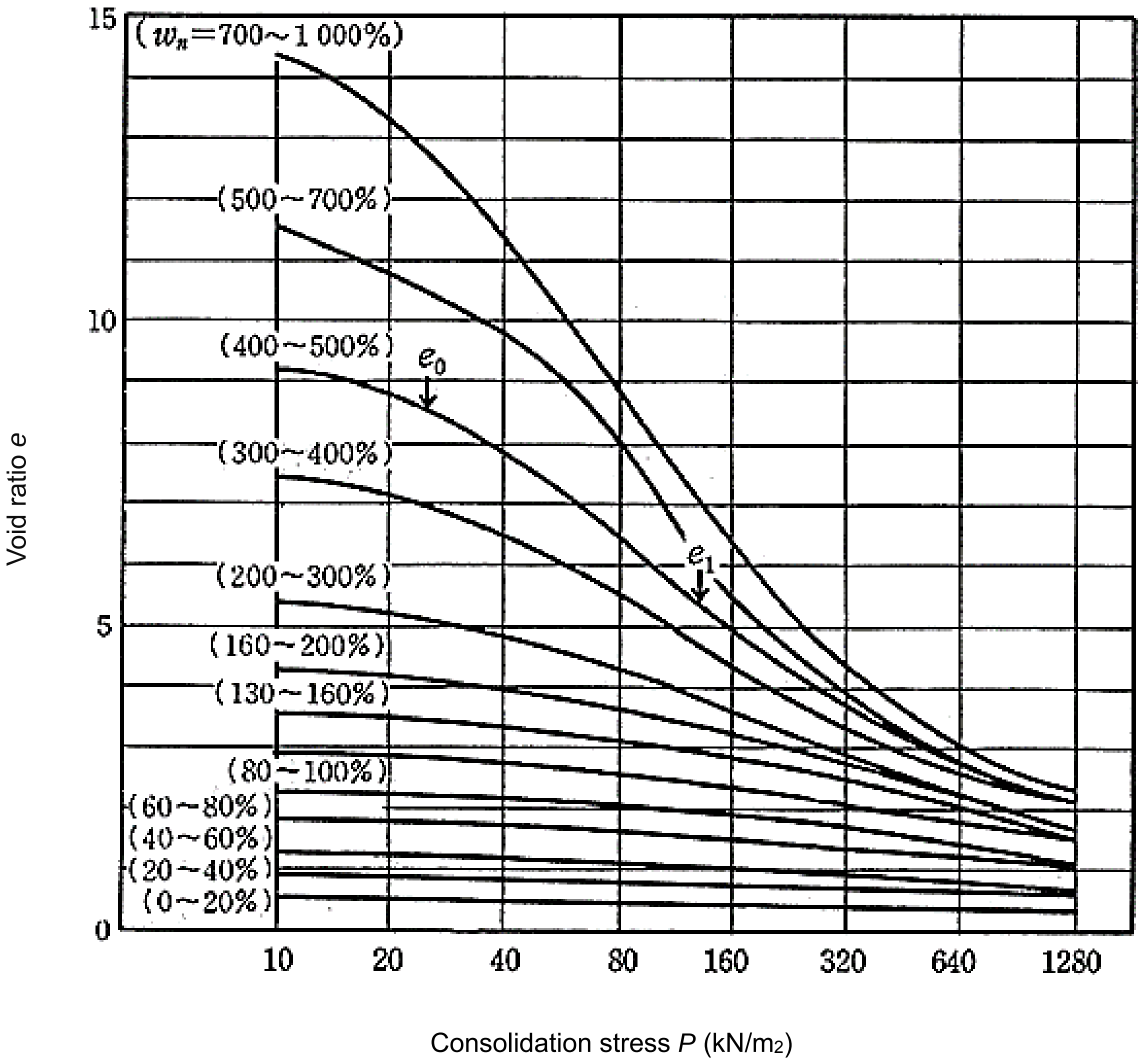


Figure 3-3 Relationship between natural water content ratio w_n and e-$\log p$ curve

e-$\log p$ Curves

The curve of a specimen that shows a value close to the average void ratio of each layer can be taken as the representative curve for the e-$\log p$ curve. The following procedure should be followed to obtain a more accurate value.

e-$\log p$ curves shown in Figure 3-4 obtained from consolidation tests of specimens taken for each depth. The representative values of e_0, p_c, and C_c as calculated from Figure 3-1 are used to draw an e-$\log p$ curve as shown by the dotted line in Figure 3-4 while observing the shapes of the curves, and this curve is taken as the representative curve.

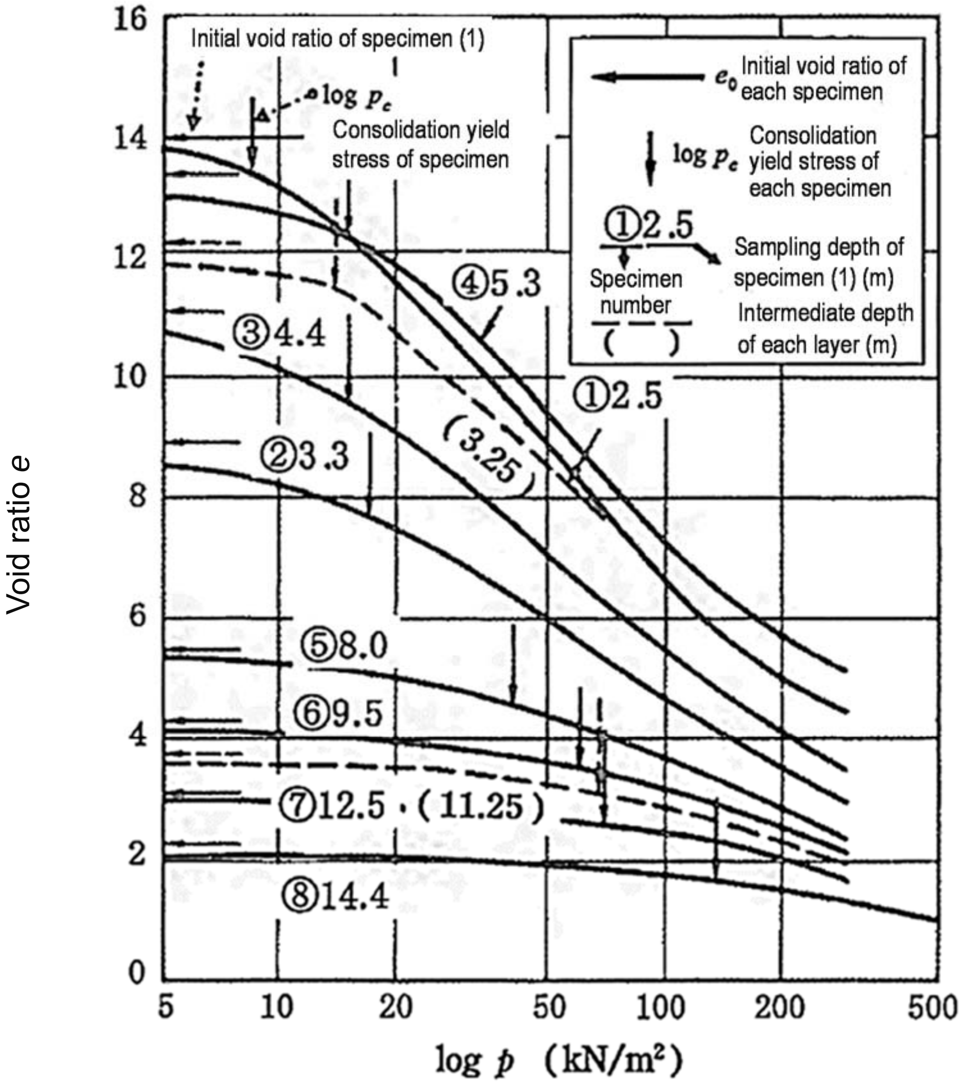


Figure 3-4 Calculation of e-$\log p$ curves representative of each layer

(4) Compression Index *Cc* obtained from Liquid Limit

The compression index C_c of a soft soil layer may be used to calculate the rough value of ground settlement. Figure 3-5 is an example that shows the relationship between the liquid limit w_L and the compression index C_c. Although the relationship between the liquid limit and compression index is different for each area, Eq.(3-1) allows us to estimate a rough value of settlement that serves as a guideline.

$$C_c = 0.009 \times (w_L - 10) \sim 0.015 \times (w_L - 19) \quad \text{Eq. (3-1)}$$

C_c *: Compression index (non-dimensional)*
w_L *: Liquid limit (%)*

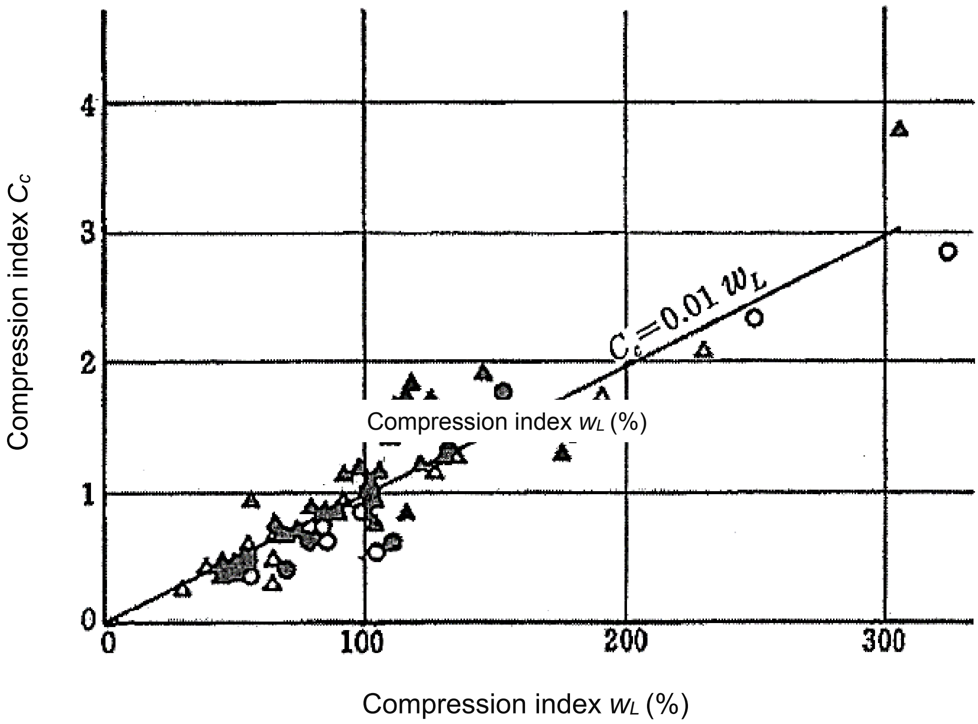


Figure 3-5 Relationship between liquid limit and compression index

(5) Coefficient of Consolidation c_v

To determine the coefficient of consolidation c_v, the $\log c_v - \log p$ curve of a specimen that shows a value closer to the average void ratio of each layer is taken as the representative curve to determine the coefficient. When the variation in the $\log c_v - \log p$ curves according to the specimen are so drastic that it is difficult to select a representative curve, the method as shown in Figure 3-6 may be used as follows.

a. draw the diagram of the vertical effective stress distribution Figure 3-6(b) after application of the embankment load in the diagram of $\log c_v - \log p$ curves Figure 3-6(a) of each specimen for comparison,

b. calculate the value of c_v that corresponds to vertical effective stress $p_0 + \Delta p/2$ after application of the embankment load at the sampling depth of each specimen,

c. sort the obtained values by depth to obtain Figure 3-6(c), and

d. calculate the average for the layers classified as shown in Figure 3-6(c) and take this as being the layer's representative c_v value.

However, caution is necessary when this method is used, because it cannot be applied to ground where consolidation is not yet finished. The data are sorted in Figure 3-6 with a certain point as an example. When soil test data are not fully available, the missing data may be estimated based on the correlation of the soil properties.

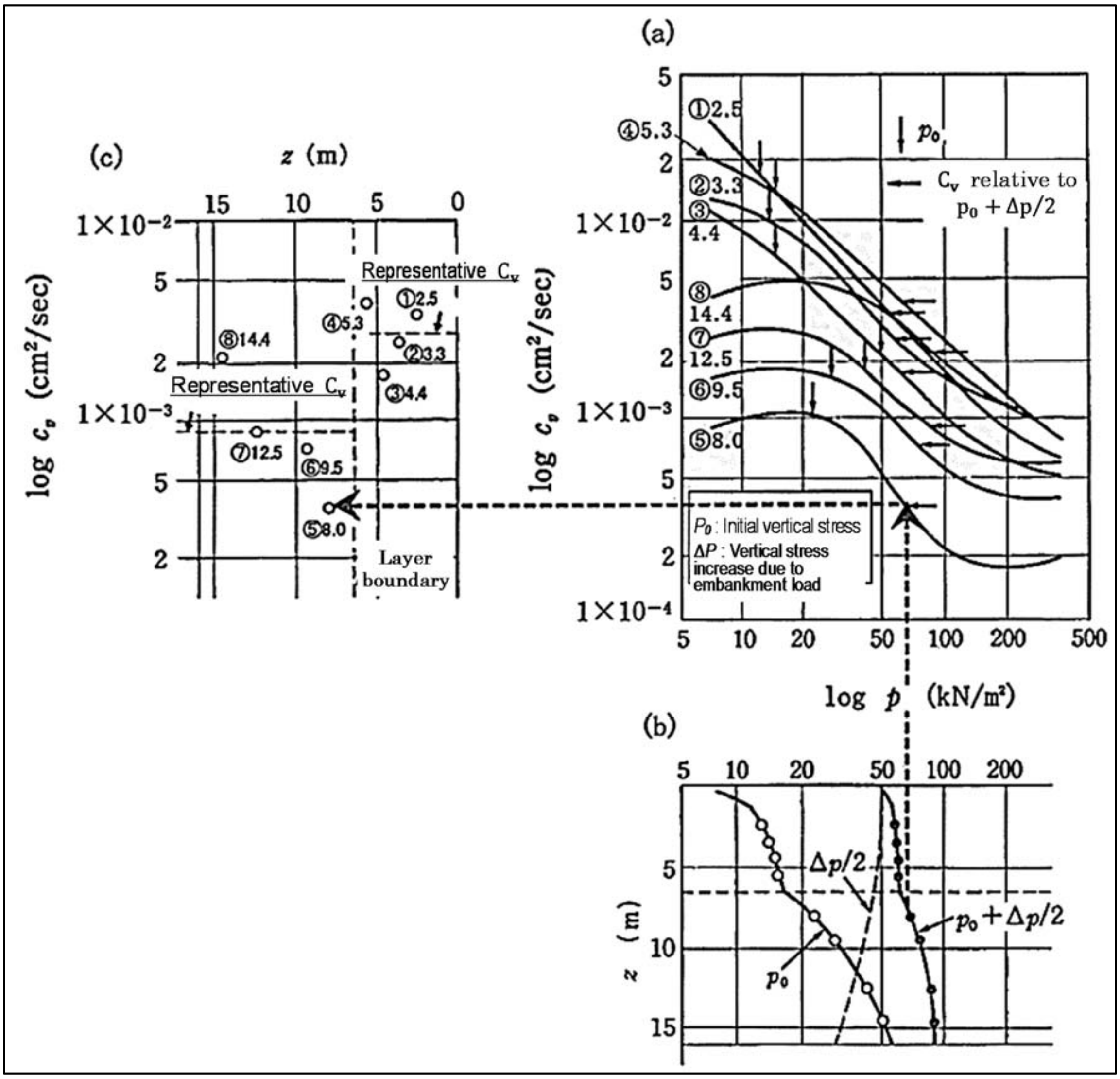


Figure 3-6 Determination of the value of representative coefficient of consolidation c_v

(6) Coefficient of Secondary Consolidation

Estimation method for coefficient of secondary consolidation

Mesri's equation (1)

$$C_\alpha = 0.0001 \times w_n \quad \text{Eq. (3-2)}$$

$C\alpha$: coefficient of secondary consolidation (1/log t)
Wn : average water content ratio of soft layer (%)

The relationship between the coefficient of secondary consolidation c_a in Eq. (3-2) and the secondary consolidation settlement rate β is described in Eq. (3-3).

$$\beta = c_\alpha \times H \quad \text{Eq. (3-3)}$$

H : thickness of the layer (cm)

Mesri's equation (2)

$$c_\alpha = (0.03\sim0.05) \times C_c \quad \text{Eq. (3-4)}$$

$C\alpha$: compression index (non-dimensional)

Estimation method by the Public Works Research Institute Japan

$$C_\alpha = 0.033 + 0.000043 \times w_n \quad \text{Eq. (3-5)}$$

(This method was developed mainly for highly organic soils in Hokkaido, Japan.)

3-3-2 Unconfined Compression Strength q_u

(1) Estimation of Unconfined Compression Strength by Soundings

Soil strength is often expressed in unconfined compressive strength q_u or adhesive force c_n (= q_n/2). When the value of q_u cannot be directly obtained from a soil test, it may be estimated from the N-value, cone index q_c, or the results of a Swedish sounding test with Eqs. (3-6) to (3-10). It is to be taken care on using an N-value obtained from a standard penetration test, because the estimated strength of N-value has large variance with poor precision, especially in the low strength range. Also, it is desirable to base the correlation equation on the empirical equation for the subject area because it depends on the soil characteristics of the area.

Standard penetration test

q_u = 25 to 50 N (kN/m^2) (N > 4) Eq. (3-6)

As per Figure 3-7 (N ≦ 4)

q_n : Unconfined compressive strength (kN/m^2)
N : N-value from the standard penetration test

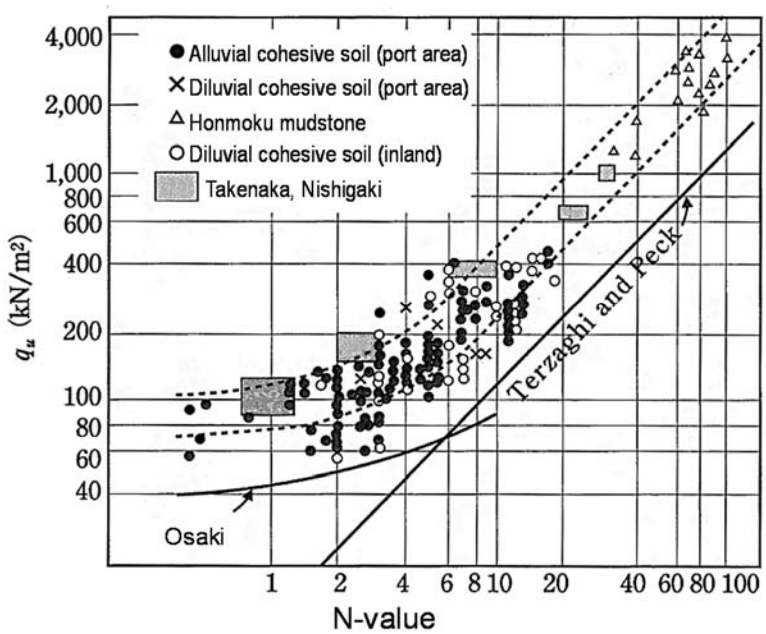


Figure 3-7 Relationship between unconfined compressive strength q_u and N-value

Cone penetration test

$$q_u = 1/5\ q_c \qquad \text{Eq. (3-7)}$$

$$= 2(q_t - \sigma_w)/N_{kt} \qquad \text{Eq. (3-8)}$$

The cone resistance q_c from a portable cone penetration test and the cone resistance q_{cd} from a Dutch double-tube static cone penetration test have the following relationship:

$$q_u \fallingdotseq 0.741 q_{cd} (\mathrm{kN/m^2}) \qquad \text{Eq. (3-9)}$$

q_c : *Cone resistance from the portable cone penetration test (kN/m^2)*
q_t : *Edge indentation resistance from the electric cone penetration test (kN/m^2)*
σ_w : *Vertical total components from the electric cone penetration test (kN/m^2)*
N_{kt} : *Cone factor from the electric cone penetration test (N_{kt} = 8–16)*
q_{cd} : *Cone resistance from Dutch double-tube static cone penetration test (kN/m^2)*

Swedish sounding test

$$q_u \fallingdotseq 0.045 W_{sw} + 0.75 N_{sw} \qquad \text{Eq. (3-10)}$$

W_{sw} : *Weight under 1,000 N (N)*
N_{sw} : *Number of half-rotations per 1 m penetration when the soil is penetrated by rotation after the penetration stopped at the above load (rotations/m)*

(2) Relation to Water Content

For the water content ratio w_n and unconfined compressive strength q_u of the ground, draw a diagram of their respective depth distributions and calculate the relationship between the average water content ratio $\overline{w}_n$ and the average unconfined compressive strength $\bar{q}_u$ for the soft soil layer H as shown in Figure 3-8. Figure 3-9 is an example of the relationship between $\overline{w}_n$ and $\bar{q}_u$ calculated as described above. Figure 3-9 allows us to calculate the average unconfined compressive strength from the average water content ratio.

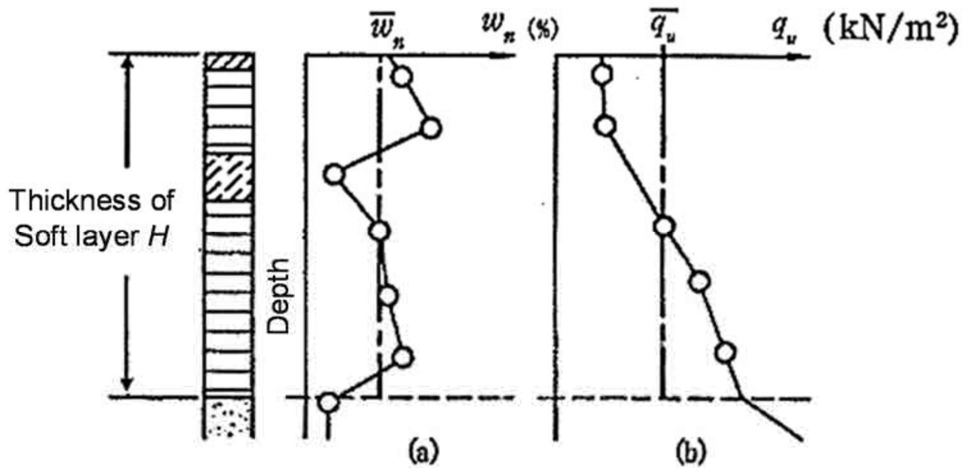


Figure 3-8 Obtaining average water content ratio $\overline{w}_n$ and average unconfined compressive strength $\bar{q}_u$

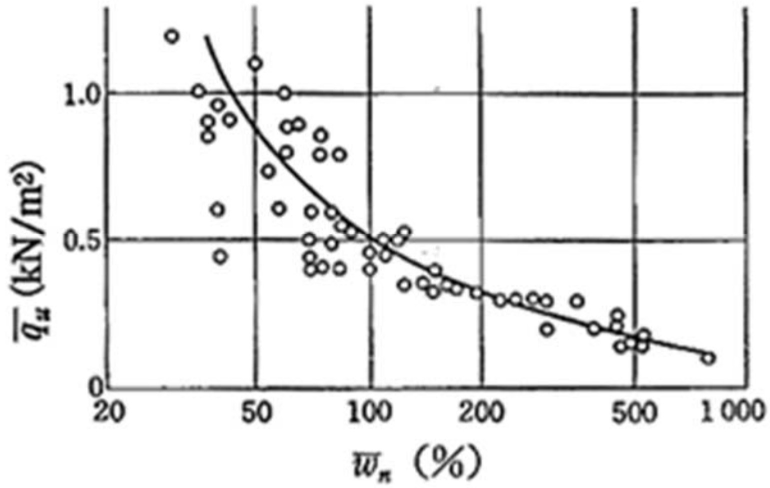


Figure 3-9 Relationship between average water content ratio $\bar{w}_n$ and average unconfined compressive strength $\bar{q}_u$

3-3-3 Shear Strength Ratio *m*

The density and strength of soil increase with the progress of consolidation. Conventionally, the strength increase ratio m is calculated using the following methods.

Triaxial compression test and direct shear test

The strength increase ratio m of normally consolidated soil is calculated as the ratio of the non-drained shear strength in a natural condition c_u to the effective overburden pressure received p_0', or c_u/p_0', as shown in Figure 3-10. However, when a design is actually developed for soft ground, it is usually necessary to know the strength increase ratio m of lightly over-consolidated soft ground. In such cases, the strength increase of over-consolidated ground is extremely small until the consolidated effective stress reaches the pre-consolidation pressure p_c' as shown in Figure 3-10. Therefore, when the ground is over-consolidated ground, the increase in non-drained shear strength due to consolidation should be ignored with respect to the consolidated effective stress under the pre-consolidation pressure, or an appropriate increase coefficient corresponding to the size of the consolidation load relative to the pre-consolidation pressure should be chosen. When determining these values in soil tests, while box shear tests provide a value close to the field stress condition (consolidated condition K_0), a triaxial compression test under isotropic consolidation conditions can produce an over-estimated value for in-situ strength, and appropriate correction should be performed.

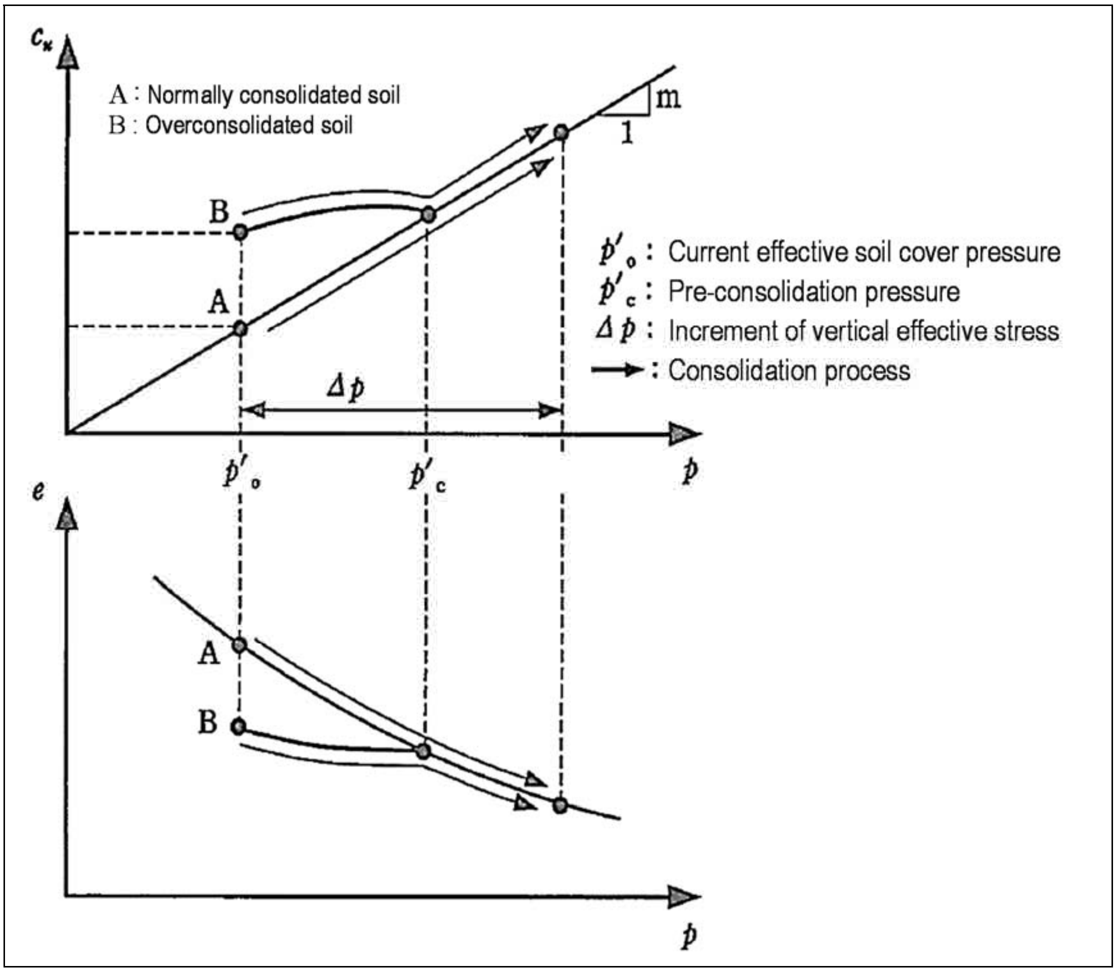


Figure 3-10 Relationship between $c_u - p$ and $e - p$

Empirical values

In many cases, soils deposited on soft ground in Japan ordinarily have a plasticity index in the range of 30 to 100. Therefore, the values given in the ranges of Table 3-5 can be used as a guideline for the strength increase ratio according to the soil properties if the undrained shear strength of the ground, the consolidated condition of the ground, the current overburden pressure, soil disturbance during works, and other relevant factors are properly considered.

Table 3-5 Guidelines for shear strength ratio *m*

Soil	*m*
Cohesive soil	0.30–0.45
Silt	0.25–0.40
Organic soil and muck	0.20–0.35
Peat	0.35–0.50

Skempton's relation equation

Method using Skempton's relation equation

$$c_u/p_0 = 0.11 + 0.0037 I_p$$

depending on the plasticity index of the soil I_p. Caution is required when applying this equation, as it is said that it is not applicable to Japanese soil.

3-3-4 N-values of Sand and Sandy Layers

In order to provide a guideline for evaluation of liquefaction or study of anti-liquefaction measures, N-values of loose sand layers or sandy soil layers with an N-value of less than about 15 are plotted as shown in Figure 3-11 to determine the representative value.

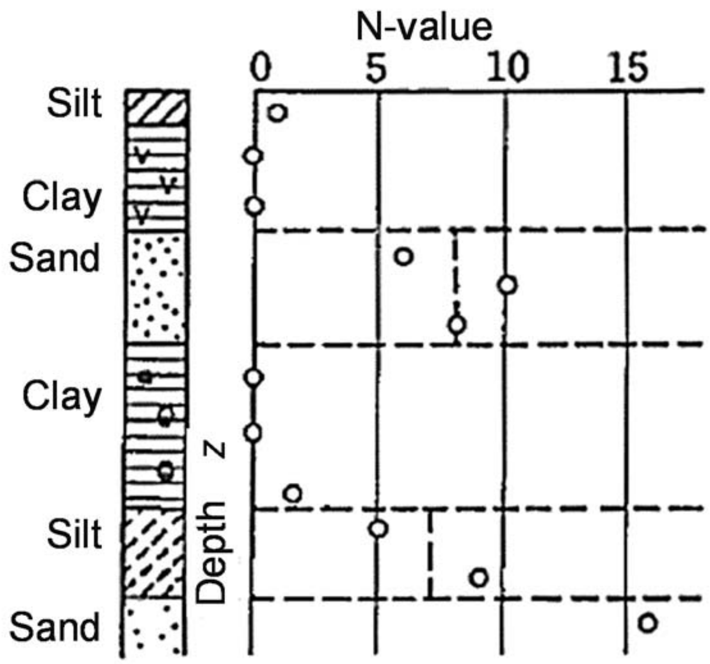


Figure 3-11 N-values representative of soil layers

3-3-5 Elastic Modulus

The elastic modulus of a cohesive soil layer is often determined from an unconfined compression test. In this operation, a secant elastic modulus E_{50} that passes a 50% point of maximum compressive stress in an unconfined compression test is used for plotting as shown in Figure 3-12 to determine the representative value. The value of deformation obtained from E_{50} of unconfined compression test is generally larger tendency. If a specimen in an unconfined compression test is seriously disturbed, compression yield strain ϵ_f become large and E_{50} become small, eventually producing excessive deformation. Therefore, it is necessary to plot compression yield strain ϵ_f as shown in Figure 3-12 to check the disturbance of the specimen and to appropriately determine E_{50}.

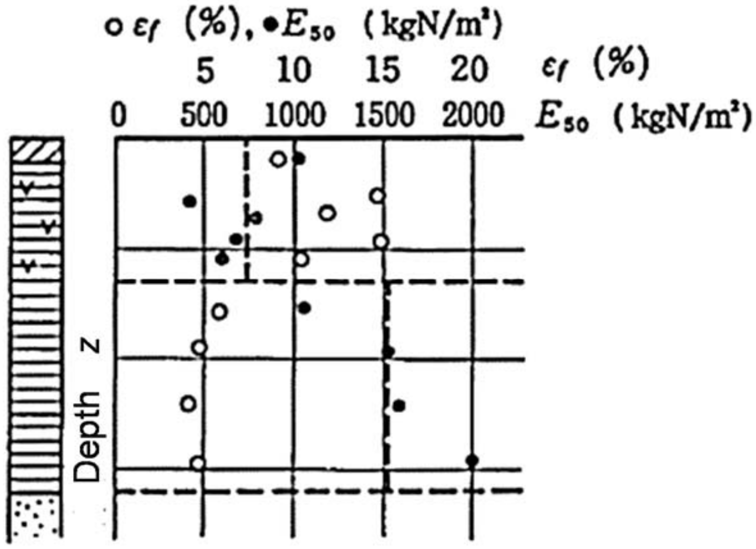


Figure 3-12 Representative values of elastic modulus E_{50} and compression failure strain ϵ_f of a soil layer

Natural water content and elastic modulus of cohesive soil

Although the value of the elastic modulus greatly differs depending on the soil, it also greatly varies even in the same soil because of its natural water content w_n. However, the elastic modulus of a soil is susceptible to the influence of disturbance during sampling and can show a smaller value. Figure 3-13 is the relationship between natural water content w_n and elastic modulus E_{50} of soils sampled from soft ground in and around Tokyo.

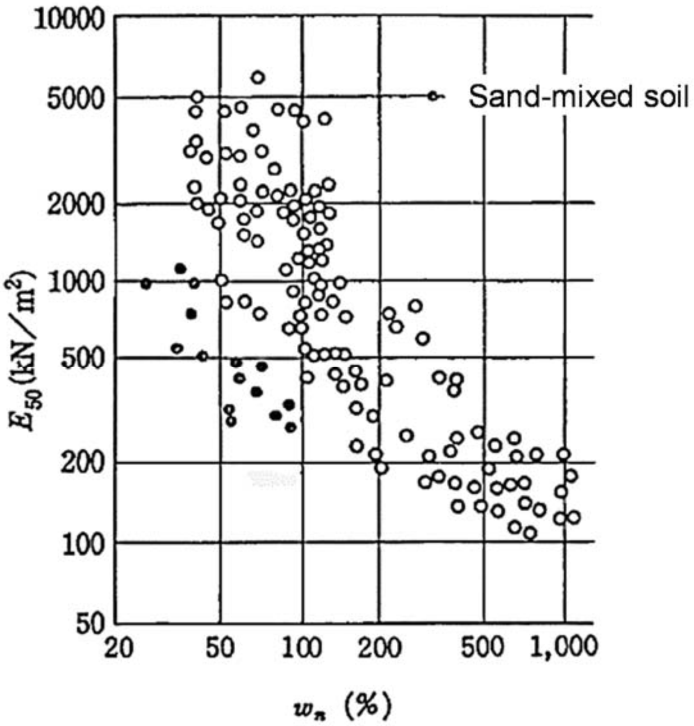


Figure 3-13 Relationship between natural water content w_n and elastic modulus E_{50} (soft ground in and around Tokyo)

3-4 Settlement Analysis

There are two categories of settlement of soft ground under the load of an embankment: one is immediate settlement that ends relatively quickly after loading, and the other is consolidation settlement that occurs slowly over a long time. Immediate settlement is further divided into settlement due to deformation resulting from shearing of a cohesive soil layer and settlement resulting from compression deformation of a loose sandy soil layer. Consolidation settlement is composed of primary consolidation and secondary consolidation. Checking needs to cover both total settlement and residual settlement. To predict residual settlement, it is necessary to calculate the rate of settlement due to consolidation, which refers to changes over time in final total settlement and settlement of the ground due to consolidation. Although secondary consolidation is taken into consideration for residual settlement, it may be ignored if the value is small. The basic theory on consolidation is based on a uniform soil layer. For actual non-uniform soil, the calculation methods of consolidation settlement ordinarily used Equivalent-thickness method or Layer-specific equivalent-thickness method in which the ground is regarded as consisting homogeneous soil layers. In recent years, the finite element method is also used, because it can better represent changes over time or spatial distributions, for example: the conditions of an embankment with a non-standard shape, the ground permeability coefficient k, the coefficient of consolidation c_v, or the volume compressibility coefficient m_v. The finite element method is used not only for primary settlement but also for deformation analysis with multiple dimensions, including deformation of the ground in the lateral direction. For details, refer to "3-2-4(3) Finite element method".

3-4-1 Allowable Settlement

The settlement analysis on normal actions is performed to make sure that settlement predicted during the construction of an earthwork structure on soft ground and during road service period does not exceed the target settlement used in the design. For checking settlement due to normal actions, the first check index is the residual settlement of the earthwork structure after the completion of pavement or opening to services. On deciding the tolerance for the residual settlement, in the design stage it is necessary to study the function of the earthwork structure, type of approach structures, the influences of settlement on road facilities, easiness of maintenance, etc. Appropriate methods are selected, either by countermeasures during construction works or by maintenance during road service period. In most cases, the design tolerance for the residual settlement is between 10 and 30 cm at the center of the embankment after the completion of pavement or in 3 years during road service period. Settlement during works is not included in the design tolerance, because it is usually possible to meet with it in the work stage. However, residual settlement and deformation of the surrounding ground are closely related to the total settlement of the earthwork structure. If total settlement is excessively large, it will seriously affect residual settlement and deformation of the surrounding ground and require a larger volume of earthwork. Therefore, it is sometimes necessary to review actions to mitigate total settlement before reviewing residual settlement. The target value for total settlement is generally decided by taking into account the amount of procurable embankment materials and the influence on the surrounding ground or adjacent facilities. The target value for the surrounding ground is properly decided with reference to “3-2-4 Deformation of the Ground around Earthwork Structures”. Although settlement of the foundation ground and settlement of the top of the embankment caused by compression of the embankment itself are the check targets, checking of the latter kind of settlement may be omitted providing that materials with little compressibility (as shown in “Road Earthworks: Embankment Work Guidelines”) are used and that the compaction management standard values are satisfied. However, if an embankment material that has a problem with compressibility is used, appropriate correction is made by using past empirical values or trial construction, and residual settlement of the earthwork structure together with settlement due to ground deformation is predicted.

The settlement of soft ground under an embankment load is composed of shear deformation, primary consolidation, secondary consolidation of a soft cohesive layer, and compression of a loose sandy layer. The amount of residual settlement after the completion of embankment is used to check the performance of the embankment against its purpose of use, and to do this, it is necessary to determine primary consolidation and secondary consolidation of the soft cohesive layer. When the secondary consolidation value is estimated small, it is usually not considered. On the other hand, the amount of total settlement, including shear deformation of soft cohesive layers and settlement and immediate settlement of sandy layers, is necessary to check the deformation of the surrounding ground as discussed in “3-2-4 Deformation of the Ground around Earthwork Structures”. The calculation method for each component of settlement is described below.

3-4-2 Basic Information for Settlement Analysis

(1) Ground Conditions

Ground conditions are determined based on the results of the preliminary investigation and as specified in “2-4 Soil Report” and “2-3-7 Verification of Test Results”.

(2) Embankment Conditions

Embankment load and pavement load

The embankment load and pavement load are determined based on the conditions in the detailed road design. It is recommendable to add a suitable load for the amount of settlement (weight in water when the level is below the groundwater level) as the embankment load. For a low embankment, the embankment load is studied by adding the traffic load calculated with the method shown in “3-2-2 (8) Settlement of Low Embankment”. The dead load of the embankment material may be calculated by multiplying the damp unit weight of the embankment material γ_t by the cubic volume of the embankment (including the pavement section). In general, Table 3-3 can be used for estimation of unit weights. Compaction tests are carried out if obviously light materials (e.g., *Shirasu* volcanic ash and sand deposits) or light-weight embankment materials are used, or if it is necessary to check the stability of a steep-sloped embankment, and the unit weight corresponding to the appropriate degree of compaction is used.

Vertical stress increment in ground due to embankment load

The vertical stress increment in ground with an embankment load Δ_p is calculated at a depth at the center of each layer as in the case of the calculation of overburden pressure. For an ordinary trapezoid-shaped embankment, the influence value for vertical stress is calculated using Figure 3-14, and the vertical stress increment Δ_p is calculated from Eq. (3-11).

$$\Delta_p = I \cdot q_E = I \cdot r_E \cdot H_E \quad \cdots\cdots \text{Eq. (3-11)}$$

Δ_p : *Vertical stress increment in the ground due to the embankment load (kN/m²)*
q_E : *Embankment load (kN/m²)*
H_E : *Embankment height (m)*
γ_E : *Unit weight of embankment (kN/m³)*
I : *Influence value (Calculate the influence value I_1 and I_2 for the left and right embankments, respectively, using Figure 3-14, and calculate $I = I_1 + I_2$.)*

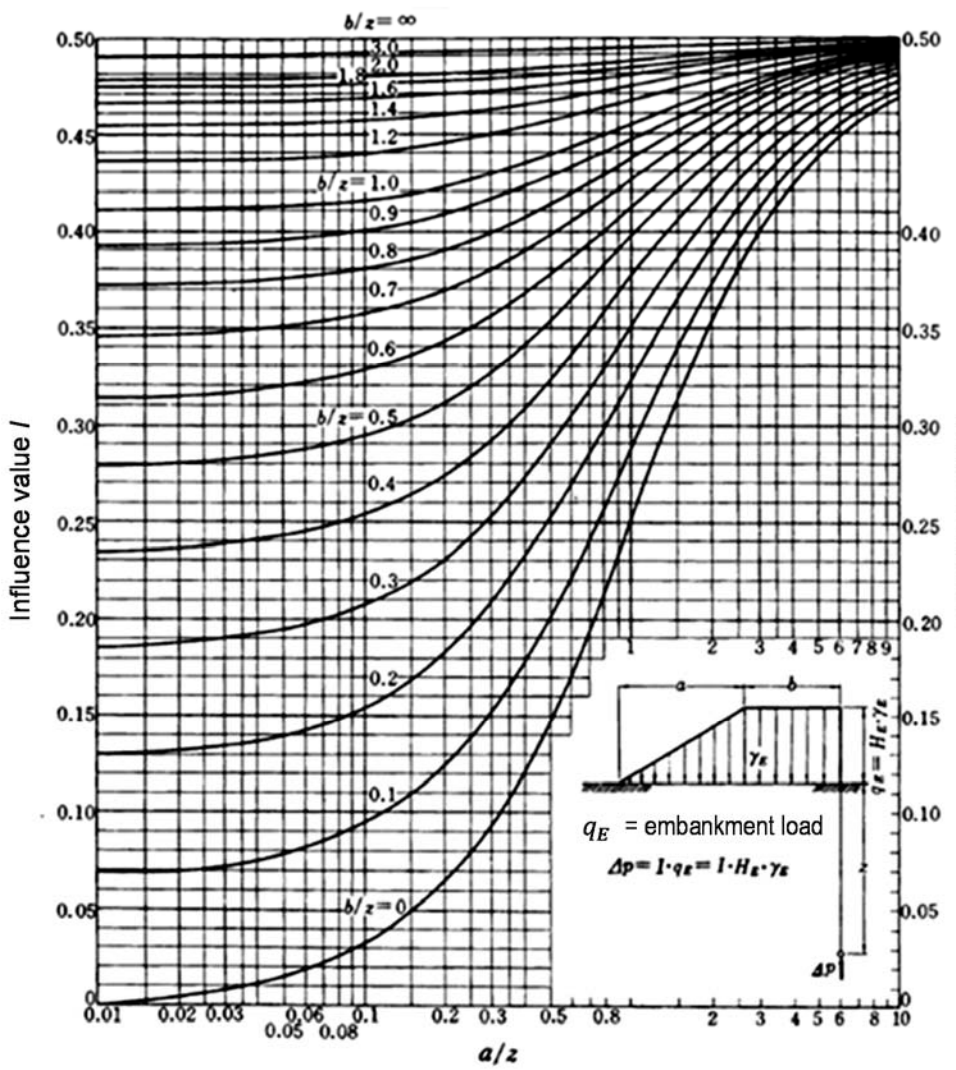


Figure 3-14 Influence value for vertical stress in the ground

3-4-3 Immediate Settlement

Immediate settlement respectively occurs in cohesive soil layers and sandy soil layers. Immediate settlement of a cohesive soil layer caused due to an embankment load is attributable to shear deformation. Because a sandy soil layer has high water permeability, its consolidation settlement is considered to be completed in a short time.

(1) Immediate Settlement of Cohesive Soil Layer

No method has been established to simply and accurately calculate the amount of immediate settlement of a cohesive soil layer. One of the methods to roughly estimate the amount of immediate settlement S_i occurring at the center of an embankment is shown below as Eq. (3-12).

<strip loading>:

$$S_i = \frac{q_E \cdot B_m}{E} \cdot \mathrm{n} \quad \text{.......... Eq. (3-12)}$$

S_i : *Immediate settlement (m)*
q_E : *Embankment load (kN/m²)*
B_m : *Loading width (m)*
n : *Coefficient calculated from Figure 3-15*
E : *Average modulus of elasticity of soft soil layer (kN/m²),*

$$E = \frac{1}{\sum H_i} \cdot \left(\sum E_i \cdot H_i\right)$$

(This equation is applicable to the ground is composed of a multiple number of soil layers i.
E_{50} *is often used for the modulus of elasticity E.)*
H_i : *Thickness of each layer that constitutes a cohesive soil layer (m)*
E_i : *Modulus of elasticity of each layer that constitutes a cohesive soil layer (kN/m²)*

Immediate settlement S_i, as shown in Eq. (3-12), is the value assumed when immediate settlement is caused by shear deformation under undrained conditions.

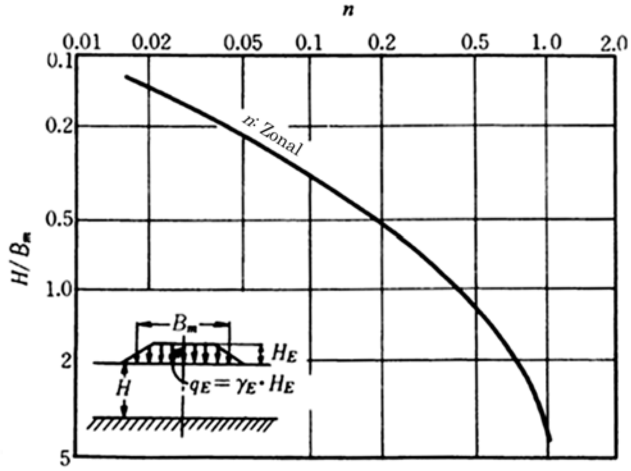


Figure 3-15 Relation between H/B_m and coefficient *n*

(2) Immediate Settlement of Sandy Soil Layer

No established equation is available for this case as well. The method described below is only an example. Principle of calculating immediate settlement occurring in a sand layer or a sandy soil layer. Settlement that occurs in a sand layer or a sandy soil layer can be calculated as immediate settlement by using Figure 3-16.

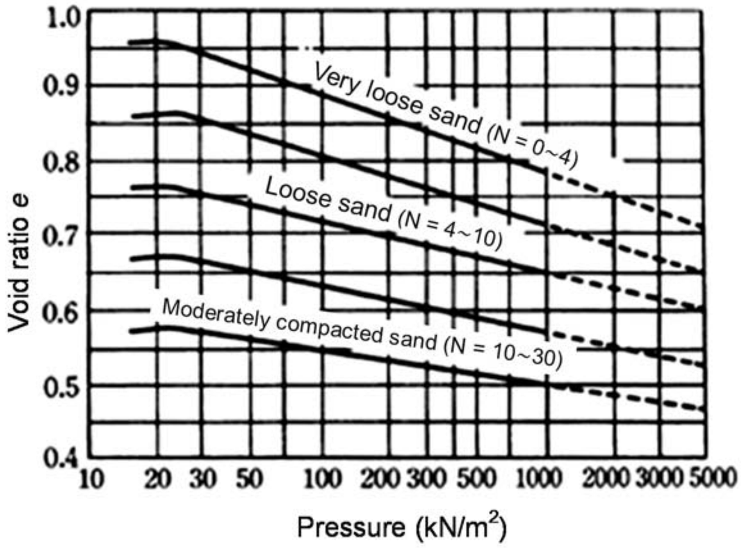


Figure 3-16 Pressure-void ratio curve of sand

3-4-4 Consolidation Settlement

(1) Settlement

To calculate primary consolidation settlement, the basic calculation is for the amount of primary consolidation settlement of a soft layer just under the center of the embankment. When it is necessary for the design, it is calculated at positions other than the center of the embankment.

The e-$\log p$ method, which is ordinarily used for calculating one-dimensional primary consolidation settlement, and the m_v and C_c methods, which are used in normally consolidated conditions, may be used to calculate primary consolidation settlement. The amount of consolidation settlement S_{cn} for each consolidated layer is summed up for the entire soft layer, and the total is the amount of settlement of the entire soft layer.

$$S_c = \sum S_{cn} \quad \cdots\cdots \text{Eq. (3-13)}$$

Method using average coefficient of volume compressibility

In this method, the average coefficient of volume compressibility $\overline{m}_v$ is calculated using the relationship shown in Figure 3-2, and the rough value of total settlement S may be calculated using Eq. (3-14).

$$S = \bar{m}_v \cdot \Delta p \cdot H \quad \text{Eq. (3-14)}$$

S : *Total settlement (m)*
$\bar{m}_v$: *Average coefficient of volume compressibility of a soft soil layer (m^2/kN)*
Δp : *Increment of vertical effective stress by embankment (kN/m^2)*
H : *Thickness of consolidated layer (m)*

Method using e- $\log p$ curve

Consolidation settlement S can be obtained by Eq. (3-15)

$$S = \frac{e_0 - e_1}{1 + e_0} \cdot H \quad \text{Eq. (3-15)}$$

e_0 : *Void ratio of initial vertical effective stress* p_0 *in the* e- $log\, p$ *curve*
e_1 : *Void ratio of* $p_1 = p_0 + \Delta p$ *in the case of an increase in vertical effective stress due to embankment loading in* e- $log\, p$ *curve*
H : *Thickness of consolidated layer (m)*

To determine a rough value of total settlement S using Eq. (3-15), e_0 and e_1 can be estimated from the relationship between the natural water content ratio w_n and the e- $\log p$ curve shown in Figure 3-3.

Method with compression index

In this method, the compression index C_c is calculated from the liquid limit w_L using Eq. (3-7), the initial void ratio e_0 at the initial vertical effective stress p_0 in the e- $\log p$ curve of the natural water content ratio w_n is calculated using Figure 3-3, and the rough value of total settlement S is then calculated using Eq. (3-16).

$$S = \frac{C_c}{1 + e_0} \cdot \log\frac{p_0 + \Delta p}{p_0} \cdot H \quad \text{Eq. (3-16)}$$

C_c : *Compression index (dimensionless)*
p_0 : *Initial vertical effective stress (kN/m^2)*
Δp : *Increment of vertical effective stress due to embankment (kN/m^2)*
H : *Thickness of consolidated layer (m)*

(2) Rate of Consolidation Settlement

The rate of settlement due to consolidation varies depending on the drainage conditions of the consolidated layer. Consolidation drainage of ground that receives a trapezoid-shaped strip load (i.e., a road embankment) is not limited to the vertical direction; drainage in the lateral direction toward the sides of the embankment also occurs at the same time. Consolidation settlement of the embankment due to so-called two-dimensional consolidation drainage will rapidly advance as the ratio of the consolidated layer thickness to the embankment width increases. In this case, as necessary, drainage in the horizontal direction is taken into account by means of the finite element method, for example. In ordinary soft ground, however, there are many cases where the thickness of a consolidated layer is much smaller than the width of the embankment due to the draining layer of an interbedded sand layer. Therefore, the settlement rate may be calculated by ignoring lateral drainage and taking the mode of drainage as one-dimensional consolidation drainage in the vertical direction only.

For calculation of the consolidation of actual ground, which often has an uneven soil composition, a draining layer (e.g., a sandy soil layer) effective for consolidation drainage is determined based on the stratification conditions of the ground, and the ground is classified into consolidated layers as shown in Figure 3-17(a) or (b). Figure 3-17(a) is an example of a consolidated layer with thickness H where the entire ground is drained on both sides, while (b) is an example of ground where the upper layer is a consolidated layer with thickness H_I drained on both sides and the lower layer is a consolidated layer with thickness H_{II} drained on a single side. For an uneven layer like this, there is a conventionally used method for calculating consolidation: (1) among the soil layers that form each consolidated layer (with the coefficient of consolidation of each of these soil layers as c_{vi}), the coefficient of consolidation of any one of the soil layers (c_{v0}) is used to represent the coefficient of the entire consolidated layer; (2) the consolidated layer is then converted to a single layer that has this coefficient c_{v0} to calculate the consolidation process. This method, called the equivalent-thickness method, is often used in actual calculations. In Figure 3-17(a), when the coefficient of consolidation of H_3, or c_{v3}, is taken as the representative coefficient of consolidation, then the equivalent thickness H_0 is calculated by Eq. (3-17) as follows.

$$H_0 = H_1\sqrt{\frac{C_{v3}}{C_{v1}}} + H_2\sqrt{\frac{C_{v3}}{C_{v2}}} + H_3 \quad \text{Eq. (3-17)}$$

H_0 : *Equivalent thickness with* c_{v3} *taken as the representative coefficient of consolidation (cm)*
H_i : *Thickness of each soil layer (cm)*
c_{vi} : *Coefficient of consolidation of each soil layer (cm^2/day)*

With this equation, a consolidated layer composed of three layers with different values of c_v, as in Figure 3-17(a), is represented by c_{v3}, and the thickness H_0 of a layer converted to a single layer with the equivalent coefficient of consolidation is given. According to Terzaghi's theory of consolidation, when pore water pressure Δu_0 occurs over the entire layer at a constant value due to immediate loading of the increment Δp of vertical effective stress, and consolidation advances with draining on both sides as in Figure 3-17(a), half of the equivalent thickness H_0 determined from Eq. (5-7) is the maximum drainage distance of the consolidated layer D. Therefore, the time t taken for consolidation can be calculated by Eq. (3-18) using the converted single layer $c_{v0} = c_{v3}$.

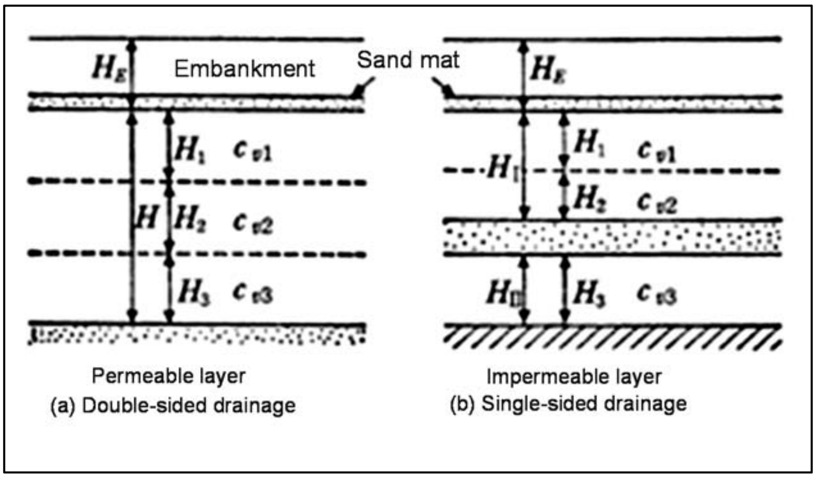


Figure 3-17 Classification of a consolidated layer

$$t = \frac{(H_0\ /\ 2)^2}{C_{v0}} \cdot T_v = \frac{D^2}{c_{v0}} \cdot T_v \quad \text{Eq. (3-18)}$$

T_v *: Time factor, which indicates the dispersion over time of excess pore water pressure* Δu_0 *(=* Δp*) that occurs at a constant value in the consolidated layer due to immediate loading of the vertical stress increment* Δp*; values shown in Figure 3-18 can be used depending on the average consolidation density U of the entire consolidated layer.*

c_{v0} *: Representative* c_v *of the consolidated layer, which is* c_{v3} *in Eq. (3-17).*

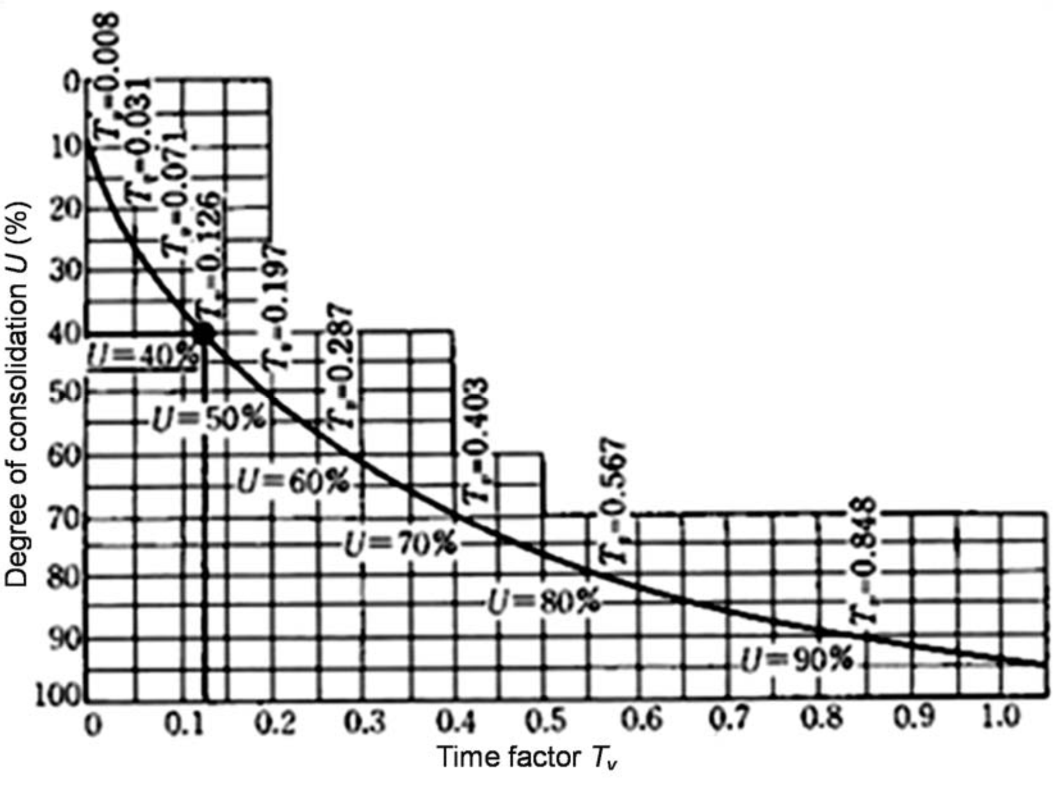


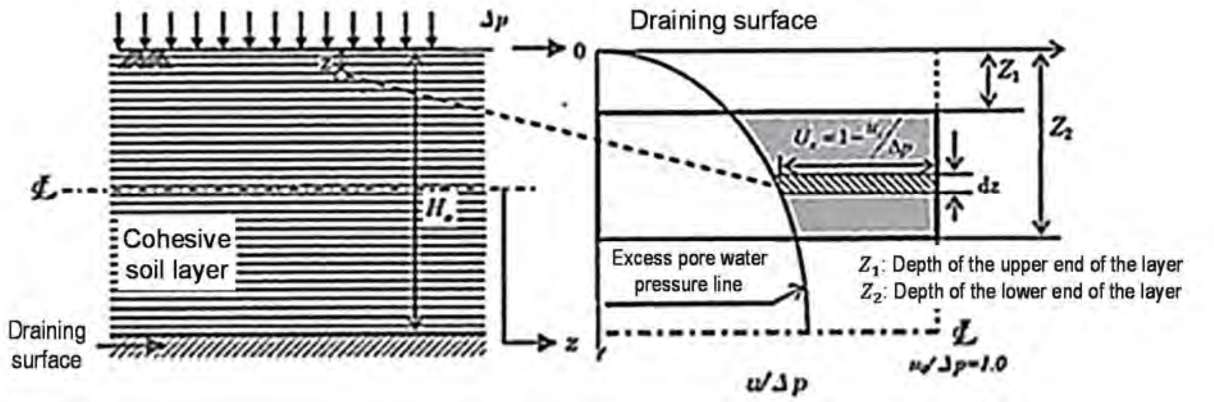
Figure 3-18 Relationship between average degree of consolidation *U* and time factor T_v for the entire consolidated layer

Consolidation settlement S_t has a relation with the total consolidated layer $\sum S_{cn}$ of each layeroliconsolidation settlement S_{cn} when the average degree of consolidation of the total layers becomes U_t. This means that time t and settlement S_t have a relation as shown in Eq. (3-19).

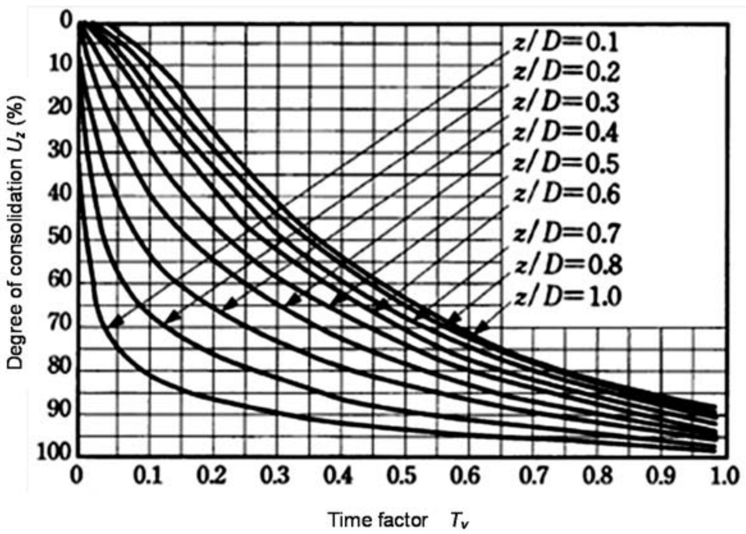
$$S_t = U_t \cdot \sum S_{cn} \quad \text{Eq. (3-19)}$$

As explained above, the rate of settlement due to consolidation can be calculated by the equivalent-thickness method with one-dimensional consolidation, assuming drainage in the vertical direction only.

However, for soft ground composed of soil layers that are remarkably different in their coefficient of consolidations c_v or coefficients of volume compressibility m_v, in order to acquire a more accurate rate of settlement, consolidation for each depth is calculated from the degree of consolidation U_z and primary consolidation settlement S_z at a depth z at a time factor T_v using Figure 3-19. Then total settlement is calculated by adding up the settlement values for each layer. When calculating the degree of consolidation U_z at a depth z in the consolidated layer relative to the average degree of consolidation U_t at a time factor t for the entire consolidated layer, select a U_z to T_v curve relative to the depth of the consolidated layer z/D from Figure 3-19, and calculate U_z relative to a time factor T_v at a point in time t calculated from Eq. (3-18).



(a) Definition of degree of consolidation U_z for each depth



(b) $U_z - T_v$ curve

Figure 3-19 Relationship between the degree of consolidation U_z at depth z and time factor T_v

Taking primary consolidation settlement of each consolidated layer as S_z, the amount of settlement S_t of the entire consolidated layer at the time point of the average degree of consolidation U_t is calculated by Eq. (3-20).

$$S_t = \sum (U_z \cdot S_z) \quad \text{Eq. (3-20)}$$

Therefore, calculating the average degree of consolidation U_n of each layer of the consolidated layer at the average degree of consolidation U for the entire consolidated layer, the amount of consolidation settlement S_t can be determined by Eq. (3-21).

$$S_t = \sum (U_n \cdot S_n) \quad \text{Eq. (3-21)}$$

S_n *: Primary consolidation settlement of each soil layer*

The flow of calculation for the equivalent-thickness method and consolidation settlement analysis in layered soils as explained above is shown in Figure 3-20. Flow "A" in Figure 3-20, primary consolidation settlement of each consolidated layer and of the entire consolidated layer due to an embankment (burden load) is calculated. Flow "B" in Figure 3-20 indicates two methods to calculate the rate of settlement. First, determine the category of the consolidated layer (including drainage conditions, or single-sided or double-side drainage), determine the equivalent thickness H_0 and drainage distance D; and calculate the consolidation time t relative to U based on the relationship between the average degree of consolidation U and the time factor T_v as in Figure 3-18.

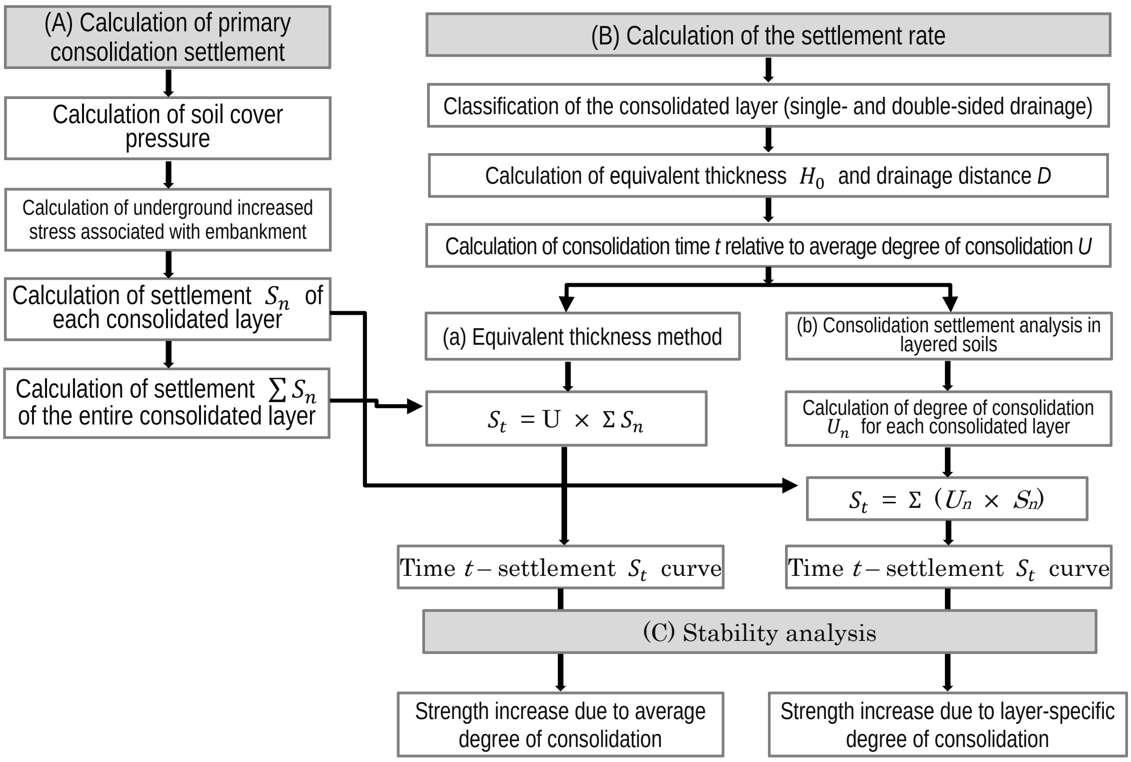


Figure 3-20 Flow of calculation for the equivalent-thickness method and layer-specific equivalent-thickness method

Then, the amount of settlement relative to the consolidation time t using the primary consolidation settlement from Flow “A” is calculated. In the equivalent-thickness method, multiply the average degree of consolidation of the entire consolidated layer U by the primary consolidation of the entire consolidated layer $\sum S_{cn}$ and calculate the settlement of the entire consolidated layer S_t. In consolidation settlement analysis in layered soils, multiply the primary consolidation settlement of each consolidated layer S_{cn} by the respective degree of consolidation U_n, add up the values for the respective layers, and calculate the settlement of the entire consolidated layer S_t. In the flow (C) stability analysis against slides are conducted. Even in this case, the degree of consolidation and the strength increase of the consolidated layer is calculated according to the equivalent-thickness method and consolidation settlement analysis in layered soils. Therefore, the stability checking results eventually turn out to be different as the evaluation of the degree of consolidation and of the strength increase for each soil layer differ between these two methods. From this point, it is necessary to study which method should be used to calculate the settlement rate. The equivalent-thickness method considers the average strength increase for the entire consolidated layer based on the average degree of consolidation, while consolidation settlement analysis in layered soils considers the strength increase for the degree of consolidation of each soil layer.

(3) Example Calculation of Consolidation Settlement

When the rate of consolidation of multiple-layer ground with different coefficient of consolidations c_v is predicted, the equivalent-thickness method is widely used, and enables a simplified calculation using the average degree of consolidation of the entire consolidated layer. However, this method does not take into account the difference in each layer’s coefficient of consolidation in the time-settlement curve, and it cannot accurately express the actual settlement process, as it produces the same degree of consolidation for either a fast-settling layer or a slow-settling layer. In addition, since the strength increase in the stability calculation depends on the degree of consolidation, the stability calculation is conducted using a different value of strength between calculation of the strength increase with the overall average degree of consolidation and with the degree of consolidation of each layer. This means that the method cannot rationally consider a layer that has a large coefficient of consolidation and a rapid increase in strength, like a peat layer, in the stability calculation. Therefore, when ground is composed of consolidated layers with greatly different coefficient of consolidations and volume compressibility coefficients (i.e., peaty ground), it is recommendable to use consolidation settlement analysis in layered soils, which takes into account the degree of consolidation according to the layer.

Calculation conditions

For an embankment load of 55 kN/m^2 applied to peaty ground, which is a 15 m thick double-side-drained consolidated layer composed of three soil layers with greatly different coefficient of consolidations, like the ground shown in Table 3-6 and Figure 3-21(a), the consolidation settlement time curve is calculated by the equivalent-thickness method and consolidation settlement analysis in layered soils as in the calculation examples below.

The properties of each layer constituting the entire ground and primary consolidation settlement S_c and equivalent thickness H_0 are shown in Table 3-21. Using the coefficient of consolidation of an organic soil layer as the representative coefficient c_v, the conversion calculation gives a uniform layer with a thickness of 11.43 m as follows:

$$H_0\left(=\sum H_{0n}\right) = H_1 \cdot \sqrt{\frac{c_{v3}}{c_{v1}}} + H_2 \cdot \sqrt{\frac{c_{v3}}{c_{v2}}} + H_3 = 100 \cdot \sqrt{\frac{194}{2380}} + 400 \cdot \sqrt{\frac{194}{2380}} + 1000 = 1143\text{cm}$$

Hence, the following relationship holds between the time for consolidation t and the time factor T_v:

$$t = \frac{(H_0\ /\ 2)^2}{c_v} \cdot T_v = \frac{571^2}{194} \cdot T_v = 1681T_v \text{ [days]}$$

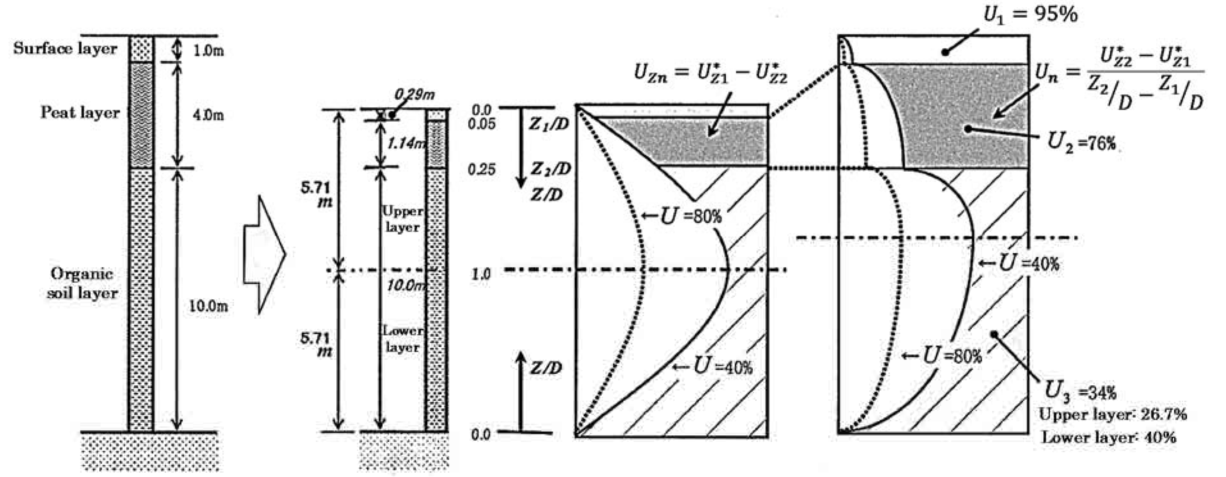
Table 3-6 Soil properties and primary consolidation settlement S_c, equivalent thickness H_0

Soil layer	H (cm)	W_n (%)	e_0	P_0 (kN/m^2)	C_c	S_c (cm)	C_v (cm^2/day)	H_0 (cm)
Surface layer	100	100	2.5	3.1	0.8	29	2,380	29
Peat layer	400	400	8.0	11.6	5.3	179	2,380	114
Organic soil layer	1,000	130	3.6	43.5	1.2	92	194	1,000
Σ	1,500	-	-	-	-	300		1,143

Calculation method and results

In consolidation settlement analysis in layered soils, the distribution of excess pore water pressure (consolidation isochrones) is drawn relative to the average degree of consolidation U_i in the entire consolidated layer, as shown in Figure 3-21(c), using the U_z–T_z relationship in Figure 3-19, in the consolidated layer converted by the standard c_v value shown in Figure 3-21(b). Then, focusing on each layer as in Figure 3-21(d), the degree of consolidation U_n for each consolidated layer is calculated. The U_n of each soil layer is multiplied by primary consolidation settlement S_{en} to obtain the result of consolidation settlement of each soil layer relative to the average degree of consolidation U of the entire consolidated layer. Following Eq. (3-21), these results are summed up for the entire consolidated layer to obtain the consolidation time and settlement of the entire consolidated layer. These results are shown in Table 3-7. This table also shows the amounts of settlement relative to the average degree of consolidation by the equivalent-thickness method.

The comparison of the time-settlement curves calculated as described above from the equivalent-thickness method and consolidation settlement analysis in layered soils is shown in Figure 3-22. In this case, the rate of settlement by consolidation calculated with the latter method is faster than that calculated with the former.



(a) Before layer thickness conversion (b) After layer thickness conversion (c) Equivalent-thickness method (d) Consolidation settlement analysis in layered soils

Figure 3-21 Isochrones of pore water pressure (consolidation isochrones)

Table 3-7 Examples of degree of consolidation, time, and settlement

			Degree of consolidation U	20	40	60	80	90
			Time factor T_v	0.031	0.126	0.286	0.567	0.848
			t=1683T_V (days)	53	211	481	953	1,425
Multi-layer analysis method	Surface layer	S_{c1} = 29cm	U_1	92	95	98	100	100
			$U_1 \times S_{c1}$ (cm)	27	28	29	29	29
	Peat layer	S_{c2} = 179cm	U_2	56	76	85	93	96
			$U_2 \times S_{c2}$ (cm)	100	137	152	166	172
	Organic soil layer	S_{c3} = 92cm	U_3	14	34	56	78	89
			$U_3 \times S_{c3}$ (cm)	13	32	52	72	82
	Σ	ΣS_{cn} = 300cm	$\Sigma U_n \times S_{cn}$	140	197	233	267	283
Equivalent-thickness method	Settlement relative to average degree of consolidation U, ($S_t = U_t \times \Sigma S_{cn}$ (cm))			60	120	180	240	270

Simplified calculation method of degree of consolidation U_z of each consolidated soil layer in consolidation settlement analysis in layered soils

In consolidation settlement analysis in layered soils, when the average degree of consolidation U_n of soil layers located at depths z_1 to z_2 in the consolidated layer relative to a given average degree of consolidation U is calculated, it is necessary to perform detailed calculation of the degree of consolidation U_{z1} to U_{z2} relative to depth ratios z_1/D to z_2/D from Figure 3-19 and sum them up. Thus, a considerable amount of time and work is required. In this case, the degree of consolidation can be simply calculated by using Figure 3-23. In this figure, the degree of consolidation at a depth ratio z/D relative to a given average degree of consolidation U determined from Figure 3-19 is accumulated in the direction of depth from the top of the consolidated layer as U_z.

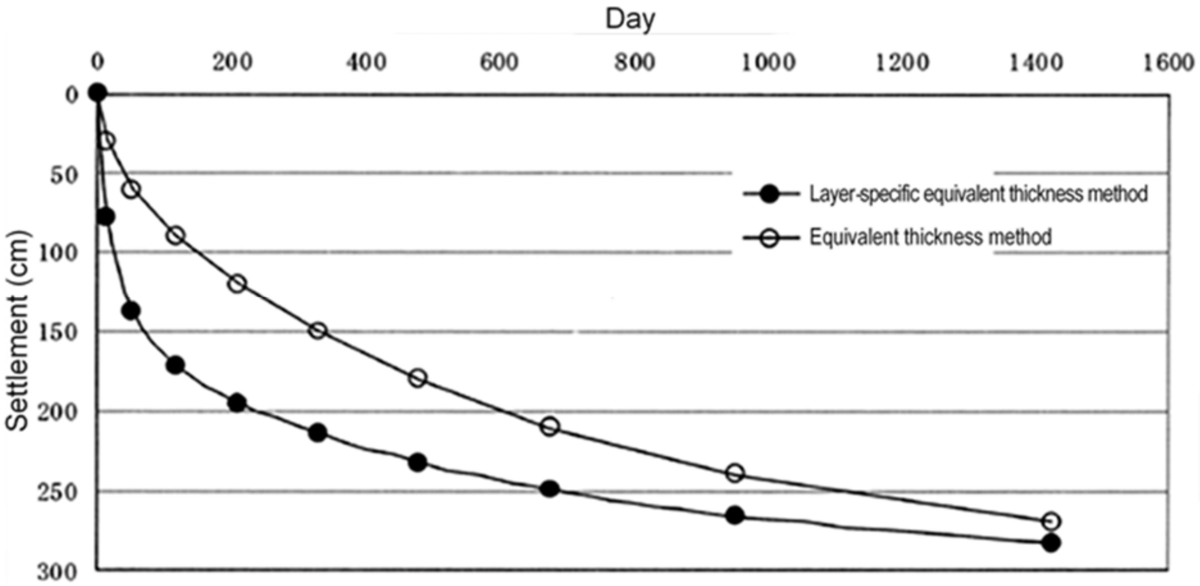


Figure 3-22 Time-settlement curves

Therefore, the cumulative degree of consolidation of soil layers between z_1 and z_2 can be obtained as the difference between the cumulative degree of consolidation $U^*{}_{z1}$ and $U^*{}_{z2}$ corresponding to depth ratios z_1/D and z_2/D. The average degree of consolidation U_n for soil layers between z_1 and z_2 corresponding to the average degree of consolidation U can be simply calculated from Eq. (3-22).

$$U_n = \frac{U^*_{z2} - U^*_{z1}}{z_2 / D - z_1 / D} \quad \text{Eq. (3-22)}$$

An example of calculating the degree of consolidation U_n of a surface layer, a peat layer, and organic soil at the time point of the average degree of consolidation U= 40% given in Table 3-7 and Figure 3-22 is explained in Table 3-8.

The calculation method of U_z of an organic soil layer given in Figure 3-21 is explained below. Since an organic soil layer is drained both in the upward and downward directions, the layer is divided into an upper and lower layer at the boundary z/D = 1.0, and the value of $U^*{}_z$ is calculated relative to z/D, as in the use example given in Figure 3-23. As shown in Figure 3-21(b), as z/D is in a range between 0.25 and 1.0 in the upper layer, the cumulative degree of consolidation $U^*{}_z$ corresponding to each z/D is calculated as 20.1% and 40%, respectively, from the line of the average degree of consolidation U= 40%. Then, using these values, the difference in the cumulative degree of consolidation of the upper layer of the organic soil is $\Delta U^*_z = 20(\fallingdotseq 40\text{-}20.1)$%. Likewise, the difference in the cumulative degree of consolidation in the lower layer is $40(40-0)$%. Thus calculated, the cumulative degree of consolidation of the upper and lower layer is the cumulative value of all the layers of the consolidated layer including the surface layer and peat layer. The average degree of consolidation U_n for the organic soil layer is obtained as 34% by dividing it by its thickness ratio ($\mathrm{U_n} = (20+40)/(0.75+1) =$ 34%).

Table 3-8 Degree of consolidation U_n of each soil layer at U = 40%

		z/D	U_Z^*	U_n
Surface layer		0.0	0	-
		0.05	4.75	-
		0.05 (=0.05-0.0)	4.75 (=4.75-0)	4.75/0.05=95%
Peat layer		0.05	4.75	-
		0.25	20	-
		0.20 (=0.25-0.05)	15.25 (=20-4.75)	15.25/0.20=76%
Organic soil layer	Upper part	1.0	40	-
		0.25	20	-
		0.75 (=1-0.25)	20 (=40-20)	20/0.75=26.7%
	Lower part	1.0	40	-
		0	0	-
		1.0 (=1.0-0)	40 (=40-0)	40/1.0=40%
	Entire layer	1.75 (=1+0.75)	60 (=20+40)	60/1.75=34%)

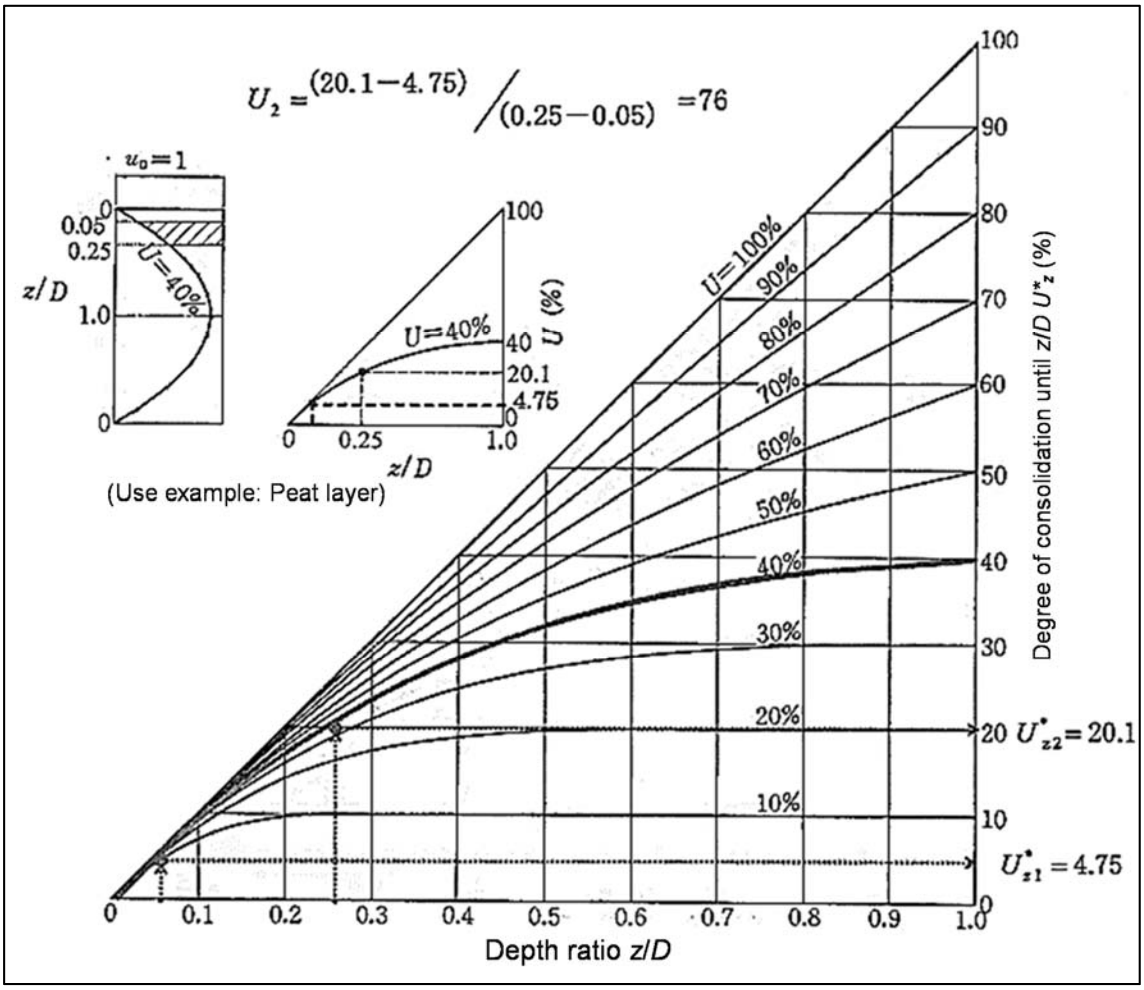


Figure 3-23 Diagram for layer-specific degree of consolidation

(4) Settlement Curve Modification based on Filling Rate

According to consolidation theory, pore water pressure changes and the process of consolidation are calculated assuming the load is immediately applied. In real embankment construction, loading is applied gradually or in stages. Taking this into account, the settlement curve is corrected approximately as follows. As shown in Figure 3-24, draw a settlement curve OAF assuming the load was immediately applied at time $t = 0$. With the embankment period set to t_0, move the settlement point A relative to time $t_0/2$ horizontally to obtain point B, which is the intersection point with time t_0, and this point will give the actual amount of settlement at time t_0. For a given point in time t, move the settlement point C relative to time $t/2$ horizontally to obtain point D, which is the intersection point with the t_0 line, and determine point E from the intersection point between the straight line connecting O and D and the time t_0 line, and take the result as the amount of settlement corrected at time t.

The amount of immediate settlement caused by gradual loading can be assumed to rapidly increase closer to the critical embankment height as shown by the solid line in Figure 3-25. However, there will be no practical problem even if settlement is taken to increase in proportion to the embankment height as shown by the dotted line in the Figure 3-25. Furthermore, when the work is left for some duration of time during the embankment work period due to the phased work process, settlement is calculated for each stage of works aligned with the time axis, and the combined settlement curve is drawn according to the superposition method.

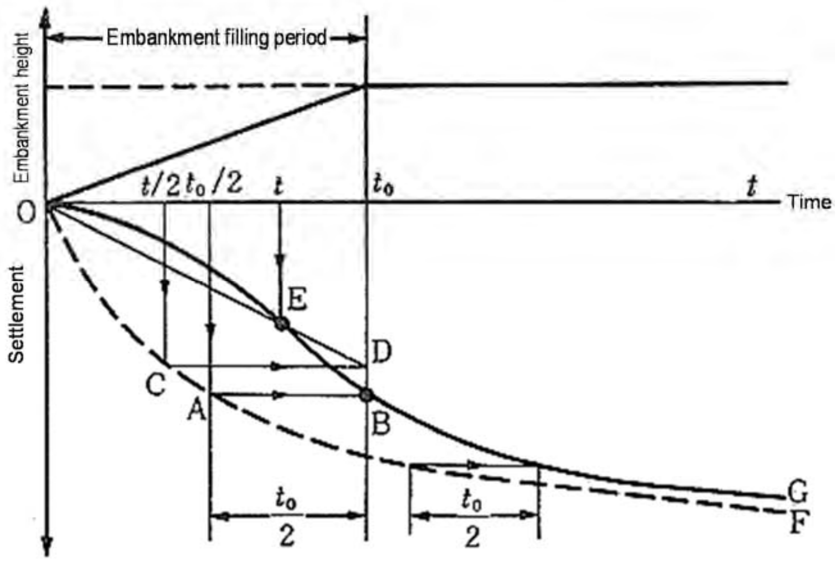


Figure 3-24 Correction of consolidation settlement curve

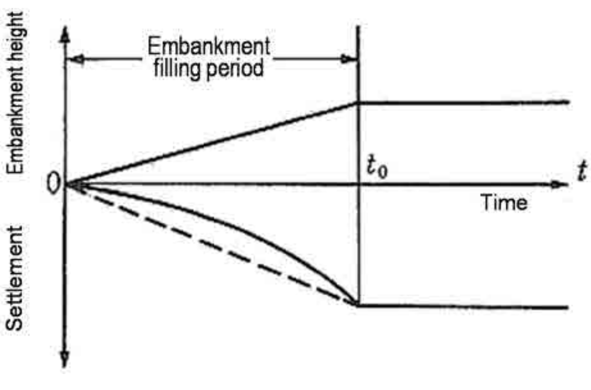


Figure 3-25 Immediate settlement by gradual loading

3-4-5 Residual Settlement

The amount of residual settlement after the completion of pavement or the road services period is also important for performance checking for road functions. Residual settlement is composed of primary consolidation and secondary consolidation after the reference time, and it is therefore recommendable to study it with secondary consolidation settlement also included. When settlement due to secondary consolidation is small, residual settlement may be ignored.

(1) Secondary Consolidation Settlement

The calculation method for residual settlement is explained below. In the design, strictly as shown in Figure 3-26, immediate settlement, primary consolidation, and secondary consolidation are studied to determine the process of settlement as a whole. Then, the difference in settlement between settlement S_t at the reference time t (e.g., opening to services) and settlement S_t at a point in time t' when residual settlement is to be determined are calculated to ultimately determine the residual settlement ΔS_t from Eq. (3-23).

$$\Delta S_t = S_{t'} - S_t \quad \text{Eq. (3-23)}$$

For estimation of residual settlement, a difference occurs in the estimation start time and the estimation period depending on the items studied. For example, there will be differences between studying the amount of camber or the extra margin of width, both of which are measures to cope with residual settlement for general embankment sections, and studying the extra margin of the cross-section of a culvert. The details on this topic is described in “4-2-6(1) Residual settlement of earth work structure”. When residual settlement ΔS_t does not satisfy the tolerance, so as to prevent negative impacts on future maintenance, necessary studies are carried out regarding changing the work schedule, the necessity of improvement work, etc.

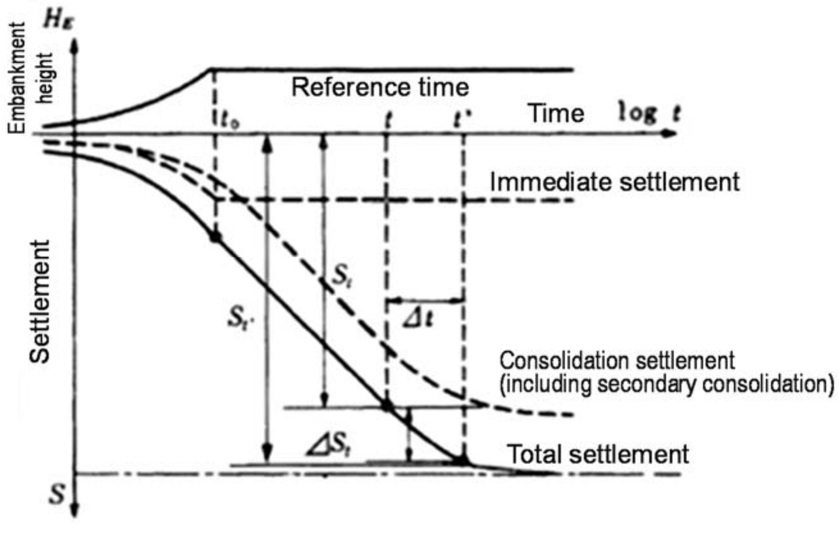


Figure 3-26 Residual settlement

Secondary consolidation settlement is large in organic soils with high water content or cohesive soils with relatively large water content, and a major problem will arise when these layers are thick. Therefore, whether or not secondary consolidation will be taken into account as consolidation in the design will be determined with consideration of the design conditions required for the road to be constructed or the soil properties of the soft ground on which the road is to be built. Regarding the secondary consolidation phenomenon, it is known from laboratory tests that settlement is often linear relative to the logarithmic time axis ($\log t$), and it has been recognized that secondary consolidation settlement continues for a long time in actual ground. However, no method has yet been established to determine the time point when settlement due to secondary consolidation is completed. Therefore, estimating residual settlement eventually requires calculating the amount of consolidation that occurs in a period determined by the design. Settlement due to secondary consolidation ΔS that occurs under a constant load after embankment is completed may be calculated by using Eq. (3-24).

$$\Delta S = \beta \times \log(t_1 \,/\, t_0) \cdots\cdots \text{Eq. (3-24)}$$

ΔS *: Secondary consolidation from time* t_0 *to* t_1 *(cm)*
β *: Secondary consolidation settlement rate (cm/log t)*
t_0 *: Number of days from the start of embankment to the start of calculation of secondary consolidation (days)*
t_1 *: Number of days from the start of embankment to the end of calculation of secondary consolidation (days)*

Considering the practical difficulty of calculating the secondary consolidation rate from laboratory tests due to the length of time involved, the relational equation on the secondary consolidation settlement rate β used in Eq. (3-24) has been proposed based on past studies on the relationship with the average water content. In a preliminary study (without detail information), residual settlement can be assumed using Eq. (3-25).

$$\Delta S = S \cdot (1 - U) \cdots\cdots \text{Eq. (3-25)}$$

ΔS *: Residual settlement (m)*
S *: Total settlement (m)*
U *: Degree of consolidation*

(2) Residual Settlement Analysis by NEXCO, Japan

Since it is practically difficult to differentiate between primary consolidation and secondary consolidation, NEXCO understands the amount of settlement to be as given in Eq. (3-26), based on the measurement results of consolidation in our expressways including the Meishin and Tomei Expressways in Japan.

$$S = S_i + S_c + S_s \cdots\cdots \text{Eq. (3-26)}$$

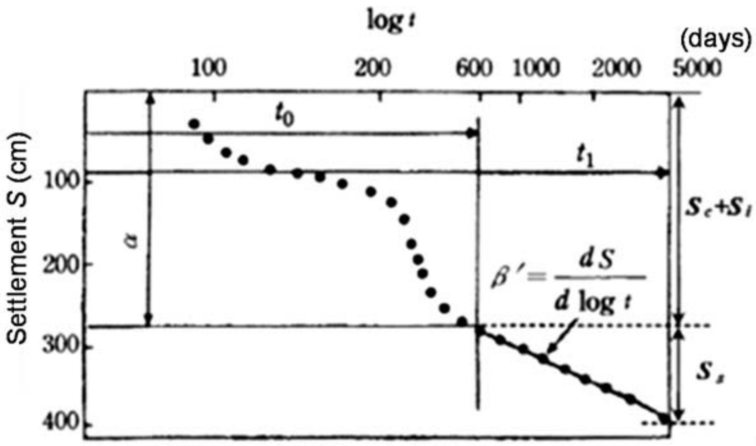
S_i *: Immediate settlement (cm)*
S_c *: Consolidation settlement (settlement until 600 days from embankment completion) (cm)*
S_s *: Long-term settlement (settlement after 600 days from embankment completion) (cm)*

Here it is assumed that consolidation settlement ends at about 600 days from embankment completion. When calculating long-term settlement, as a general rule changes in consolidation settlement over time are not taken into account, and settlement that occurs thereafter is referred to as long-term settlement. For long-term settlement occurring after about 600 days from embankment completion, there is a relationship as shown in Figures 3-27 and 5-R8 between the consolidation rate (β') and the maximum draining distance of the soft layer, and the estimation equation of long-term settlement S_s can be obtained as shown in Eq. (3-27). From this it is understood that long-term settlement S_s can be estimated with reference to past data and Figure 3-28.

$$S_s = \beta \times \log(t_1 / t_0) \quad \text{Eq. (3-27)}$$

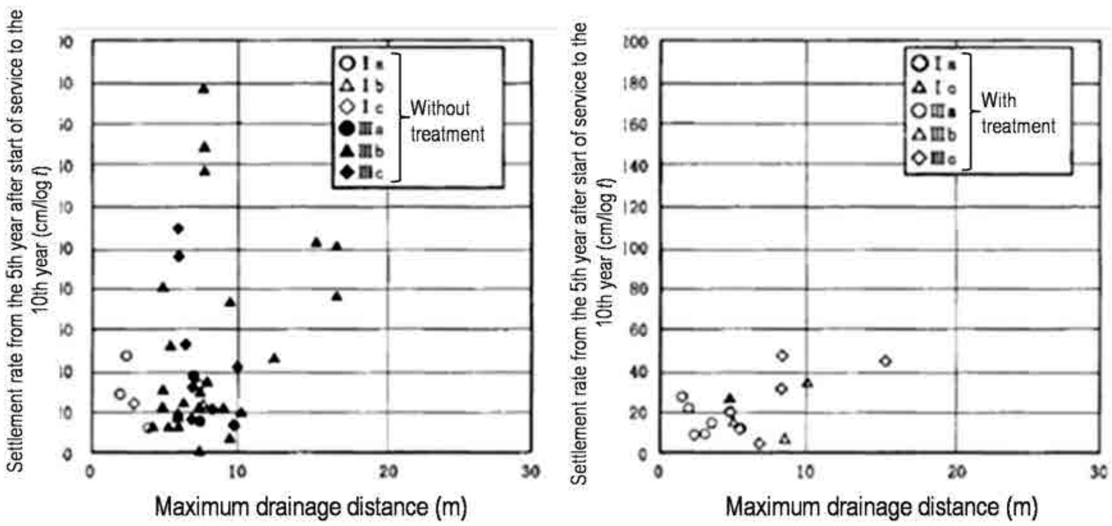
β' : Long-term settlement (cm/log t)
t_0 : 600 days after start of embankment (start time of long-term settlement) (days)
t_1 : Number of days from start of embankment for long-term settlement estimation date (days)

The estimation equation used here was developed for past embankments on expressways. Caution is necessary when the equation is applied to embankments with a notably different scale or construction method.



Note that α in Figure 3-27 is the amount of settlement (cm) at t_0 (600 days after the start of embankment).

Figure 3-27 Long-term settlement



Note: Signs in the figures are based on the classification of ground types given in Table3-9 (for example, Ib is a Type I upper sand layer).

Figure 3-28 Settlement rate and maximum draining distance during road service period

3-4-6 Settlement of Low Embankment

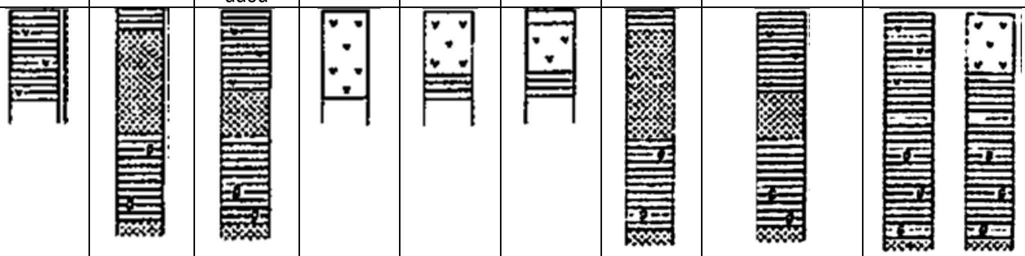
With low embankments on soft ground, there are few problems of stability or lateral deformation as in the case of high embankments, or large settlement during construction or during road service period. However, there are many cases of a phenomenon where road surface unevenness occurs during road service period and the pavement eventually fails due to the following mechanisms.

a. Since the embankment adjacent to the soft layer is low, sufficient compaction could not be completed for the subgrade section, and it is difficult to generate sufficient bearing capacity of the subgrade.

b. When the underground water level is high, because underground water rises to the vicinity of the roadbed, its bearing capacity is easily lowered.

c. The traffic loads that repeatedly act on the pavement surface are not sufficiently dispersed in the embankment and have reached the soft ground, promoting settlement deformation of the ground.

Table 3-9 Ground type classification and problems by soil composition of soft ground

Ground type		Type I			Type II (peat type)			Type III (including peaty soil)		
Maximum drainage distance of soft layer		Less than 5 m			Less than 5 m			5 m or more		
Settlement characteristics		Almost completed in six months after embankment			Tend to be almost completed in six months after embankment			Settlement continues for a long time (Review use of ground improvement work by trial embankment)		
Stability		Relatively good			Careful slow and staged construction, counterweight fill, or embankment on soft ground with net matting			Good		Same as peat type
Size of repair in the management stage		Careful response should cause no problem			Functional problems rarely caused if sufficient time is given for loading (shelf time)			Repair necessary for a long period of time over 10 years		
Schematic columnar section		I a	I b	I c	II a	II b	II c	III a	III b	III c
		Thin type	Upper sand layer	Sand layer inter-bedded	Singly peat	Peat + clay	Peat inter-bedded	Upper sand layer	Sand layer inter-bedded	Continuous Peat + clay
										
Topography, and settlement and stability	Delta back marsh							◎ △	● △	
	Inland back marsh	○ △			● ▼	◎ ▼	◎ ▼		● △	
	Lagoonal marsh				● ▼	◎ ▼		◎ △	● ▼	● ▼
	Drowned valley				● ▼	◎ ▼				● ▼
	Aggraded valley	○ ▼			● ▼	◎ ▼				● ▼
	Colluvial valley	○ △								

Note: ●: Large settlement ◎: Slightly large settlement ○: Small settlement ▼: Stability problem exists △: Only minor stability problem

There are a number of methods for predicting settlement, including estimation methods using past observation values or consolidation tests with cyclic loads applied to a sampled specimen. These methods, however, have not yet been fully established. Figure 3-29 shows an embankment load equivalent to the influence of traffic loads according to the embankment height based on a settlement curve obtained from ground monitoring of low-embankment roads. As required, an embankment load equivalent to the influence of a traffic load obtained from Figure 3-29 can be used to study settlement that occurs during road service period and the extra fill to cope with it. For example, when a low embankment of 2 m in height is planned, the embankment load equivalent to the influence of the traffic load is estimated to be about 20 to 30 kN/m^2.

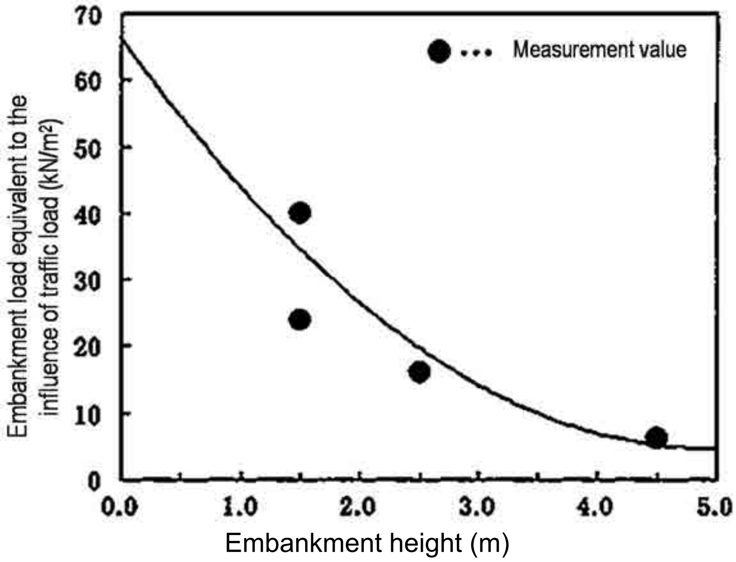


Figure 3-29 Embankment load equivalent to the influence of traffic load

3-4-7 Prediction of Final Settlement based on Settlement Monitoring

Prediction of the settlement of soft ground requires the use of appropriate estimation means suited to the target or purpose of estimation, since the available information or the required precision varies depending on when the estimation is conducted. The NEXCO Design Procedures provide rough guidelines about how to select estimation means with due consideration of the purpose or timing of an estimation, as shown in Table 3-10. Specifically, in the design stage, long-term settlement is estimated using the method explained in section 3-2-2(7)2) for general embankment sections. When studying the margin of the earthwork width or extra banking, or an extra cross-section of the crossing structure, studies are carried out using a number of the methods shown in the table. In the construction stage, the hyperbolic method or log t method based on the ground monitoring data is used to predict future settlement, and the amount of settlement planned in the design is corrected as described in “Chapter 5 Work Control and Maintenance” for details of use in the construction stage.

As already explained, predicting embankment-induced settlement is required for predicting the necessary amount of fill or for evaluating the necessity of soft ground improvement work. In the design, it is recommendable to use soil investigation or laboratory soil test results, and to make highly accurate predictions using appropriate analysis techniques or empirical techniques.

However, there are many cases where a gap occurs between what is envisaged in the design and the actual consolidation phenomena attributable to ground unevenness, or due to the limits of the applicability of analysis or empirical techniques. During embankment and after the completion of embankment, it is effective to carry out ground monitoring (e.g., the amount of embankment settlement) in order to predict what the difference between the design and actual values would be, which will make it possible to study necessary measures at an early stage. In other words, predicting the amount of embankment settlement includes predictions based on design calculations, and ground monitoring after the start of construction.

Table 3-10 Purpose and means of settlement estimation (NEXCO Design Procedures)

Purpose of estimation		Estimation time		Information	Estimation means	Remarks
		Design	Construction			
General embankment section	Settlement	○	○	Soil investigation Soil test Ground monitoring	Design: $S = S_i + S_c$ Construction: Observation with settlement plate	
	Earthwork extra banking	○	○	Ground monitoring	Design: $S = S_i + S_c + S_s$ Construction: $S = S_0 + \frac{t}{a+bt}$	Study settlement in two years during road service period
	Extra margin for earthwork width	○		Existing material	$S = \alpha + \beta \log \frac{t}{t_0} (= S_s)$	Study settlement in five years during road service period
Crossing structure	Extra banking	○ (Note)	○	Ground monitoring	$S = \alpha + \beta \log \frac{t}{t_0} (= S_s)$	Culverts S_{r1} and S_{r2} are measured values.
	Extra margin for section	○	○	Soil investigation Soil test	$S = \alpha + \beta \log \frac{t}{t_0} (= S_s)$	Culverts Equivalent-thickness method is a theoretical value.
	Preload removal period		○	Existing material Ground monitoring	$S = \alpha + \beta \log \frac{t}{t_0} (= S_s)$	Preload shelf time shall be six months or longer. This rule, however, does not apply if the soft layer is 10 m or less in thickness.
Pavement road surface	Planned height		○	Leveling	–	–
	Temporary pavement	○	○	Existing material Ground monitoring	Design: $S = \alpha + \beta \log \frac{t}{t_0}$ Construction: $S = S_0 + \frac{t}{a+bt}$	Study settlement in five years during road service period
Secondary structures	Structure of protective fence, communication piping, etc.	○	○	Soil investigation Soil test	Shall be decided based on construction time	–

S_i : *Immediate settlement*
S_c : *Consolidation settlement*
S_s : *Long-term settlement*
S_0 : *Initial settlement (t = 0) (cm)*
t_0 : *Time when settlement becomes linear in the S-log t curve*
a : *Constant determined from the measured settlement (1/cm)*
b : *Constant determined from the measured settlement (1/cm • day)*
α : *Settlement determined from past empirical values or measured settlement curve (cm)*
β : *Settlement determined from past empirical values or measured settlement curve (cm/log t)*
S_{r1} : *Settlement that has occurred up to the 600th day from embankment completion starting from burden load removal*
S_{r2} : *Settlement due to the influence of the ground's rebound upon burden load removal*
S_{r3} : *Long-term settlement occurring after 600 days from embankment completion*

3-5 Stability Analysis

3-5-1 Concept of Stability Analysis

For checking the stability of an earthwork structure on soft ground against normal actions during works and during road service period, it is necessary to check the stability of embankments against slips and of retaining walls and culverts in terms of slips, slides, bearing capacity, and overall stability. In checking of the stability of embankments against normal actions, the stability calculation method assuming a circular slip surface is used for the study. For calculating stability, the results of "3-2-2 Settlement" corresponding to the ground conditions or the filling rate are used to conduct the analysis, and the safety factor is used as the check index. In this case, the main target of a stability check is the stability of the embankment against slips upon completion of the embankment and during road service period. The recommendable safety factor is 1.10 or higher during the construction works with ground monitoring and 1.25 or higher after completion.

The following concept is used as the basis for establishing a target value for the stability and settlement of an earthwork structure in a preliminary study.

(1) Target safety factor

The target value for the minimum safety factor against a slip failure of the foundation ground caused due to an earthwork structure is 1.2–1.3. However, when the design is carried out assuming the use of instrumentation and monitoring construction, the target safety factor should be flexibly set depending on the content or precision of the monitoring systems during work period.

(2) Soil strength increase

In a preliminary study, the safety factor against ground slip failure is determined using a total stress method that ignores the strength increase due to consolidation of the foundation ground. In the detailed design stage, stability is calculated with consideration of the strength increase resulting from consolidation. In a preliminary study stage, however, sufficient soil data are not usually available, and the strength increase due to consolidation is not considered.

(3) Embankment failure

As the embankment height increases with the progress of embankment, the amount of settlement of the embankment and the amount of uplift of the surrounding ground increase. If the embankment load exceeds the ultimate bearing capacity of the ground, the embankment fails along the slip surface as shown schematically in Figure 3-30.

When an embankment is built on soft ground, primary consolidation of the soft ground during embankment is calculated by consolidation calculation, and circular slip calculation is conducted taking into account the strength increase associated with primary consolidation. The stability analysis is conducted by calculating the safety factor against slips upon completion of embankment and during road service period. The ground monitoring also is adopted in order to respond with various uncertain factors during the works. Even after the completion of embankment, it is necessary to ensure stability indicated in “3-2-3 Settlement”.

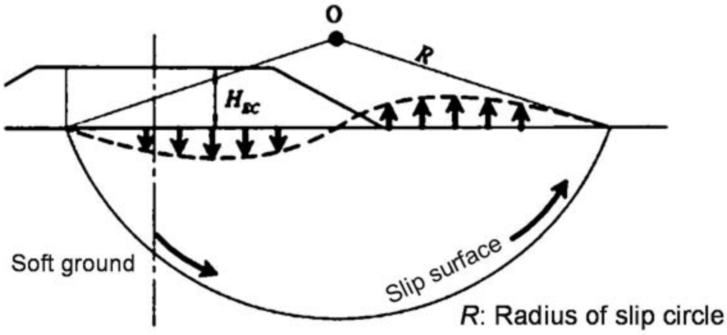


Figure 3-30 Failure of embankment on soft ground

3-5-2 Basic Information for Stability Analysis

(1) Ground conditions

Determine the ground conditions based on the results of detailed investigations and as specified by “2-4 Soil Report” and “3-1-3 Ground Condition”.

(2) Embankment Conditions

The following embankment conditions should be obtained according to the description in “3-2-2(2)1)b. Embankment condition”.

a. Embankment load and pavement load

b. Unit weight of embankment material

c. Stress in the ground due to the embankment and other loads

d. Filling rate

If the ground has settled from the original ground surface in the process of embankment, it is recommendable not to include the settled portion of the height in the embankment height providing that instrumentation and monitoring is conducted during the works. If stability calculation is carried out with the necessary embankment thickness taking into account the settled portion of the embankment height, the result will produce overestimated embankment loads. This can lead to the implementation of unnecessary measures and ultimately to producing an uneconomical design.

3-5-3 Strength Increase of Cohesive Soil

(1) Increased stress of soft layer due to embankment

Calculate the vertical effective stress increment Δp due to an embankment load in a layer whose embankment stability is affected.

(2) Degree of consolidation of soft layer

While the average degree of consolidation is often used as the degree of consolidation of a soft layer according to the equivalent-thickness method, when there is an inter-bedded layer with remarkably different consolidation characteristics, using the degree of consolidation of each soil layer U_n at the point in time when studying stability by consolidation settlement analysis in layered soils is also studied.

(3) Strength increase of cohesive soil

In the stability calculation, the safety factor against slip failure of an embankment is determined by the total stress method that uses un-drained cohesion c taking into account the strength increase associated with consolidation of the soft layer. A procedure for calculating un-drained cohesion c that ignores changes in strength in an over-consolidated region is explained below.

Un-drained cohesion c_u uses the value resulting from Eq. (3-28) with the degree of consolidation taken into account as shown Figure 3-31.

Initial condition ($P_0 = P_c'$) of normal consolidated state

$$c = c_0 + m \cdot \Delta p \cdot U \quad \text{Eq. (3-28a)}$$

Normal consolidated state reached by embankment load ($p_0 + \Delta p > p_c'$)

$$c = c_0 + m \cdot (p_0 - p_c' + \Delta p) \cdot U \quad \text{Eq. (3-28b)}$$

Over-consolidated state remains after loading of the embankment load ($p_0 + \Delta p \leqq p_c'$)

$$c = c_0 \quad \text{Eq. (3-28c)}$$

c_0 : *Undrained cohesion of soil in the original ground before embankment (kN/m^2)*
m : *Strength increase ratio (non-dimensional)(see "3-3-3 Shear Strength ratio m")*
p_0 : *Vertical effective stress of the soil layer related to the slip surface before embankment (kN/m^2)*
P_c' : *Preceding consolidation stress* $P_c' = {}^{c_0}/_{m}$ *(kN/m^2)*
Δp : *Increase in vertical stress due to embankment load occurring to a soil layer related to slip surface (kN/m^2)*
U : *Degree of consolidation of a soil layer related to slip surface*
p_t : *Vertical effective stress at degree of consolidation U (kN/m^2)*
c_{uf} : *Undrained cohesion upon completion of consolidation (degree of consolidation: 100%) (kN/m^2)*
c_{ut} : *Undrained cohesion at degree of consolidation U (kN/m^2)*

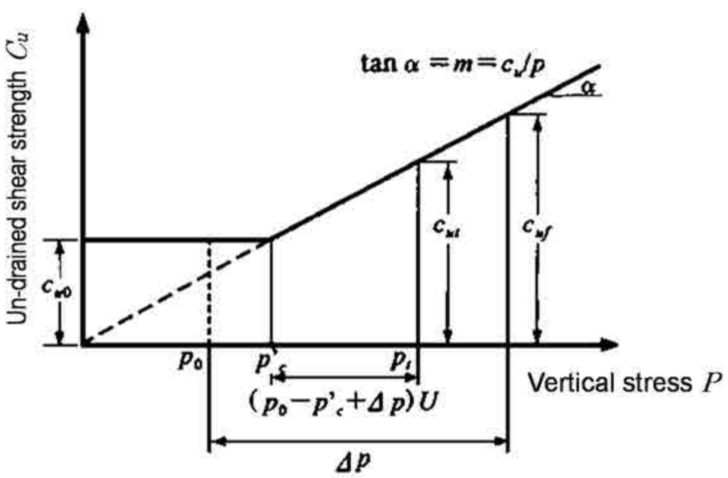


Figure 3-31 Shear strength increase due to consolidation

3-5-4 Slip Stability Calculation

(1) Method of Slip Stability Calculation

Calculating the stability of an embankment is conventionally conducted by two methods: the total stress method and the effective stress method. In principle, the effective stress method is the correct method, as the shear characteristics of soil are essentially dominated by effective stress. In order to apply this method, it is necessary to know the pore water pressure that occurs associated with the shear of the saturated soil; however, this is ordinarily difficult. Practically, it is assumed that only taking into account the pore water pressure under hydrostatic pressure is appropriate for the total stress method. For the general stability calculation procedure, a soil mass on a circular slip is divided into a number of narrow slices having a vertical profile as shown in Figure 3-32, and the safety factor F_s against slip failure of the entire soil mass is calculated. Moreover, the position of the center of the circular arc and the size of the radius are sequentially varied, and the minimum value of the safety factor is taken as the safety factor against slip failure. The calculation equation used in this operation is Eq. (3-29).

In the case of a soft cohesive soil layer, c takes the undrained cohesion c_u (kN/m^2) given by Eq. (3-28) that takes into account the strength increase due to consolidation on the slip surface of the bottom of a slice. In the case of a saturated cohesive soil layer, ϕ takes 0.

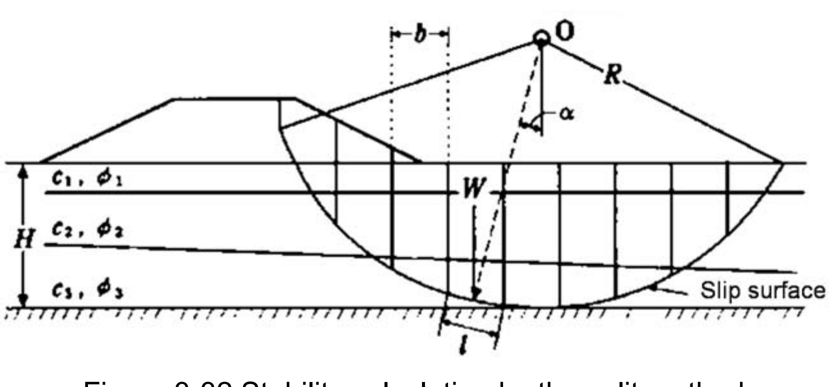


Figure 3-32 Stability calculation by the split method

$$F_s = \frac{\sum[c \cdot l + (w - u_0 b)\cos\alpha \cdot \tan\phi]}{\sum(W \sin\alpha)} \quad \text{Eq. (3-29)}$$

F_s : *Safety factor*
c : *Cohesion of soil (kN/m2)*
ϕ : *Angle of internal friction of soil (˚)*
l : *Length of the slip surface of the slice (m)*
W : *Total weight of the slice, including burden loads (kN/m)*
u_0 : *Pore water pressure (kN/m²)*
b : *Width of slice (m)*
α : *Average slant angle of slip surface of a slice (˚)*

(2) Important Points in Stability Calculation

Points in conducting the above stability calculation are described below.

Shape of slip surface

In the circular slip calculation, it is often the case that the smallest safety factor turns out to be a slip surface that goes through not only soft layers with small shear strength, but also layers whose consolidation progress is slow due to the layer thickness and coefficient of consolidation c_v conditions. While the shape of an actual slip surface often have a complicated curvature, slope analysis assuming circular form of the slip surface could not be problem.

Slip surface in the embankment

Calculation regarding the embankment is conducted taking into account the shear resistance along the assumed slip surface. When an embankment sits on thick soft ground, the embankment might receive lateral tension associated with lateral displacement of the ground. Thus, in the stability calculation, it is recommendable to ignore the shear resistance of the embankment and perform the calculations based on the assumption that vertical tension cracks occur in the embankment section. In this case, crack depth is often calculated from Eq. (3-30). For example, the NEXCO gives 2.5 m or under for the depth of a tension crack.

$$z_t = \frac{2 \cdot c}{\gamma_E} \cdot \left(45° + \frac{\phi}{2}\right) \quad \text{Eq. (3-30)}$$

Z_t : *Depth of tension crack (m)*
γ_E : *Unit weight of embankment (kN/m³)*
c : *Cohesion of embankment (kN/m²)*
ϕ : *Angle of internal friction of embankment (˚)*

Number of slices

Although it cannot be stated definitively due to the shape of the embankment's section, the soil classification of the soft layer, etc., the number of narrow slices into which a slip mass is divided is generally determined by taking into account the size of the analytical model or how complicated it is.

Construction machinery load

In some cases the working loads of construction machinery are taken into account in stability calculations. In these cases, 10 kN/m^2 is applied to the top of the embankment as the burden load, as specified by "3-1-1(3) Burden Load".

Stability calculation for high embankments

For an embankment that is relatively high for the thickness of the soft layer on which it stands, stability problems may occur before the embankment reaches the planned height. Therefore, taking into account the work process or other conditions, it is necessary not only to calculate stability against the final embankment height H_{E2} as shown in Figure 3-33, but also to study stability during embankment filling (e.g., embankment height H_{E1}, etc.).

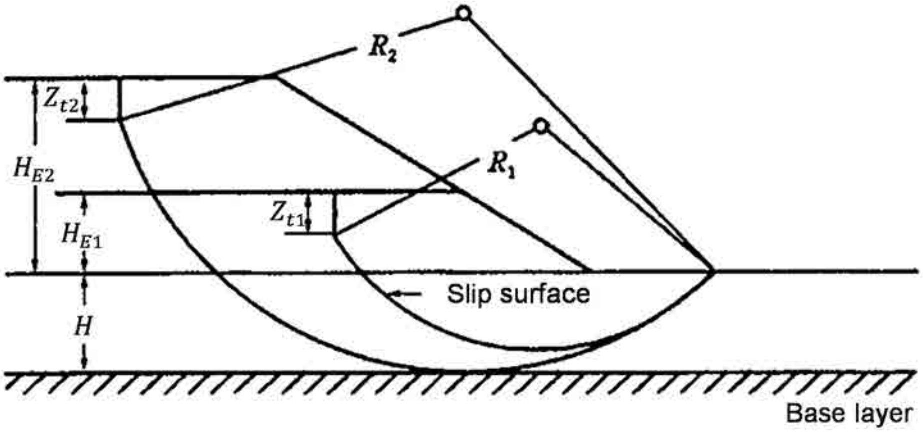


Figure 3-33 Stability calculation for high embankments

Reduction in shear strength

For a type of soil that has a high plasticity index or sensitivity ratio (e.g., silt or organic soil), its shear strength will remarkably decrease if the soil is seriously displaced or disturbed in the process of work. When an embankment is constructed on ground composed of depositions of these soils, it is recommendable to slow down the filling rate as much as possible, taking into account the disturbance of the soil due to ground improvement, local reduction in strength associated with this disturbance, or progressive failure, and to set the strength increase ratio m taking into account experimental construction or past construction records.

3-6 Deformation of the Ground around Earthwork Structures

3-6-1 Basic Concept

When an embankment is built on soft ground, there will be lateral displacement together with settlement of the ground under the embankment, and deformation of the surrounding ground will occur. Deformation of the soft ground due to the embankment load is composed of immediate settlement, uplift, undrained shear deformation, and consolidation settlement. Immediate settlement, uplift, and lateral displacement due to undrained shear deformation occur at almost the same time as loading. This is elastic deformation when the applied load is small and becomes plastic as the load increases. This type of settlement due to shear deformation occurs in a short time, and so it is also called immediate settlement. Settlement due to soil consolidation is a phenomenon that occurs with time. Soil consolidation is divided into two types, primary consolidation resulting from the dispersion of excess pore water pressure and secondary consolidation resulting from compressive creep of the clay skeleton. The latter begins to appear approximately at the end of primary consolidation, and its speed is extremely slow.

Figures 3-34(a) and (b) show ground deformation by separating it into shear deformation and consolidation deformation. In shear deformation, the ground just under the embankment is compressed in the vertical direction due to the embankment load and stretches in the horizontal direction as shown in Figure 3-34(a). This change pushes out the surrounding ground horizontally, and displacement distribution as shown in the figure occurs in the ground near the slope toe of the embankment. Consequently, the surface of the surrounding ground is uplifted and displaced horizontally toward the outer side. On the other hand, in consolidation deformation, the embankment load affects not only the ground just under it but also the surrounding ground as shown in Figure 3-34(b). As a result, consolidation occurs equivalent to the increment of stress (volumetric compression), resulting in settlement distribution as shown in the figure. The horizontal displacement distribution in the ground near the slope toe of the embankment is pulled inward, in the opposite direction to shear deformation, as shown in the figure.

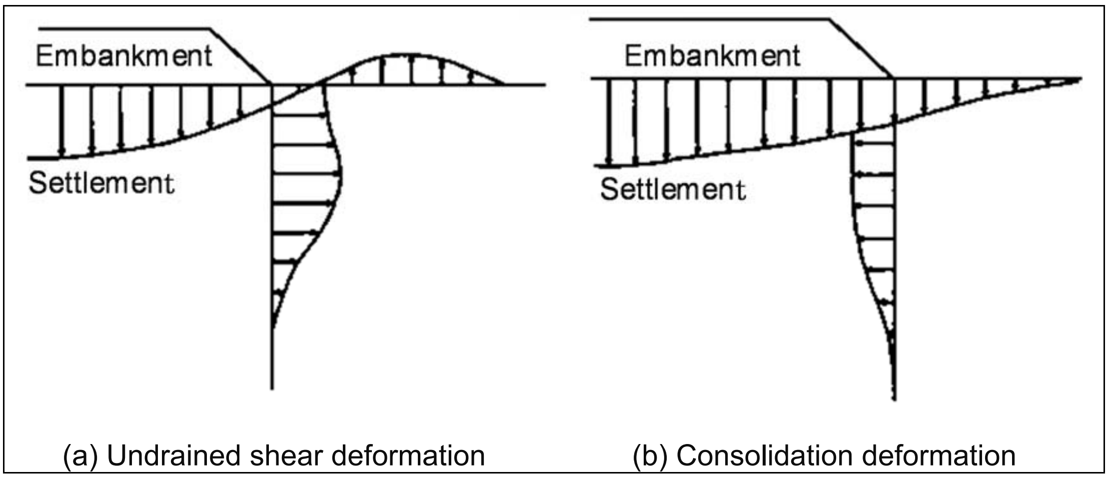


Figure 3-34 Deformation of the ground by load of embankment

Figure 3-35(a) is a schematic diagram showing undrained shear deformation of the ground due to the embankment load and settlement and uplift of the surface of the soft ground due to consolidation deformation, in the direction crossing the embankment. Figure 3-35(b) shows changes in settlement over time at the center of the embankment. The dotted lines in Figure 3-35(a) show the amount of settlement due to consolidation at the points in time t_0 and t_1, and the solid lines indicate the total settlement (or uplift) due to consolidation and shear deformation.

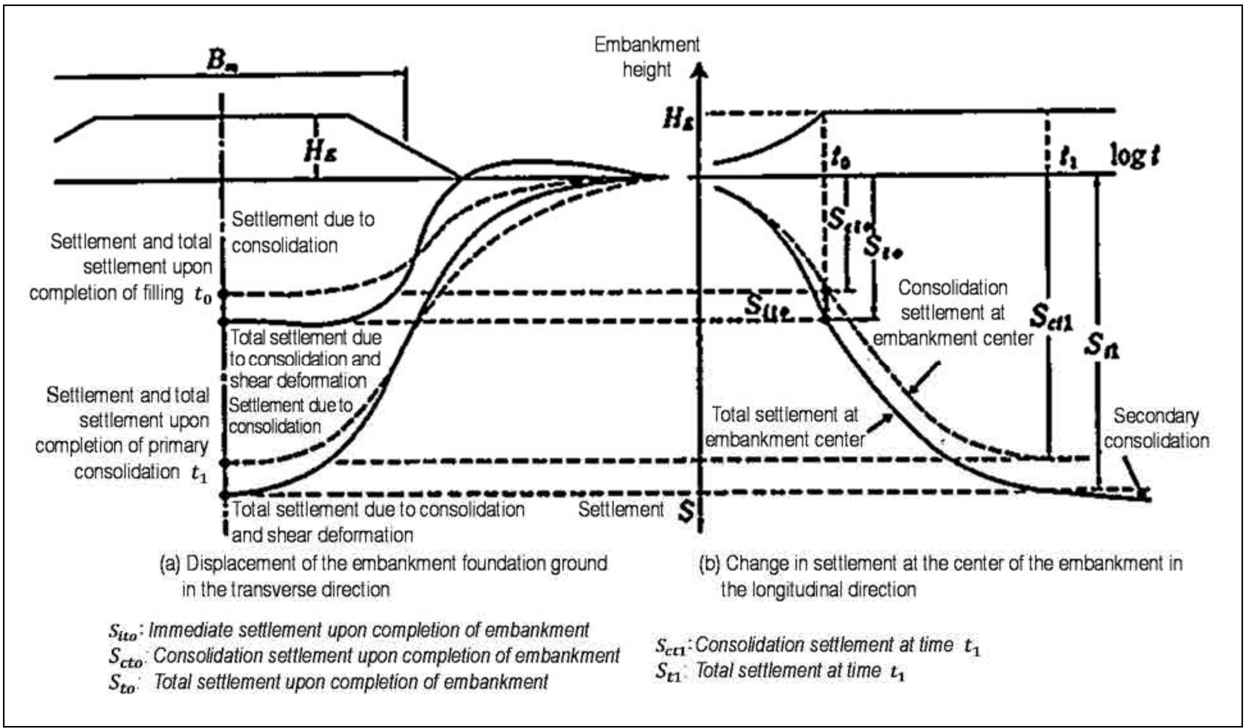


Figure 3-35 Changes in the settlement and uplift of embankment foundation ground and in settlement at the center of the embankment

In the settlement-time relation diagram for the center of the embankment as shown in Figure 3-35(b), the dotted lines show consolidation settlement, while the solid lines show total settlement, including immediate settlement due to shear deformation. Although immediate settlement or uplift due to shear deformation increases as the embankment load increases, both show almost no increase after embankment is completed. Therefore, only consolidation settlement occurs after the completion of embankment. For total settlement after the completion of embankment, as shown in Figure 3-35(a), with settlement due to shear deformation remaining almost fixed, the total amount of settlement has the approximately constant settlement shape shown by the solid line. Secondary consolidation causes settlement with an approximately constant slope against the logarithm of time ($\log t$) after the completion of primary consolidation t_1.

Other cases of embankment settlement include settlement due to traffic loads in the case of a low embankment, compression of the embankment itself, and settlement due to groundwater level changes.

3-6-2 Checking of Deformation

If the surrounding ground can suffer problem of ground deformation during construction of an earthwork structure or after it opens to services, the amount of deformation shall be predicted not exceeding the allowable displacement decided in the design. The basic purpose of checking against regular deformation is to maintain the functions of facilities in the vicinity that may be affected by deformation of soft ground. However, taking economic conditions into account, this study may be omitted if there is the potential to restore the function of the structure with leased land or the like even if its functions would be temporarily lost. Regarding these performance requirements related to regular deformation, the necessity of study or the maximum deformation with which the intended functions can be maintained are determined according to the functions and performance requirements of the target to be affected by deformation. Therefore, it is recommendable to carry out a good investigation of the characteristics and functions of the target structure and establish appropriate management standard values. In this sense, the allowable values related to deformation are determined according to the structure to be affected. In reality, however, they will be determined based on consultation with the managers of the structures.

There are various ways of deformation analysis as shown in 3-6-3, and the precision of each method will be greatly affected by the deformation prediction method or the establishment of the ground data. Therefore, it is recommendable to gather work records in advance for the nearby area that may serve as reference data.

3-6-3 Prediction of Ground Deformation

A number of prediction methods that estimate deformation based on past observation data have been proposed as follows.

(1) Settlement shape of embankment and the influence on the surrounding ground

Embankment settlement shapes and their influence on lateral sides actually observed from expressways including the Meishin and Tomei expressways and ordinary national highways are expressed in the form of Eq. (3-31), and the values of each coefficient, C_1 and C_2, are shown in Figure 3-36.

Settlement	$S_t = C_1 \cdot S$	Eq. (3-31a)
Ground uplift	$\delta_t = C_1 \cdot S$	Eq. (3-31b)
Ground lateral movement	$\mathrm{x} = C_2 \cdot S$	Eq. (3-31c)

C_1, C_2 : *Coefficients (values in Figure 3-36)*
S : *Final total settlement at the center of the embankment (m)*
H : *Thickness of soft layer (m)*
x : *Distance from the embankment (m)*

Figure 3-36 is an example of a road embankment constructed in a period of 50 to 200 days for a base width of 30 to 60 m. The figure indicates that deformation is very small at a location more than twice as far from the slope toe of the embankment than the soft layer thickness. The shape shown by the solid line in Figure 3-36 is the settlement shape generally seen in many cases upon completion of embankment. When the soft layer is thin and the rate of consolidation is fast or the embankment rate is remarkably slow, the settlement shape shown by the broken line is observed. Since deformation of the surrounding ground is deeply connected to the total settlement of the embankment or the embankment rate, controlling the influence on the surrounding ground within the tolerance will eventually require maintaining the total settlement of the embankment or the embankment rate below a certain limit level.

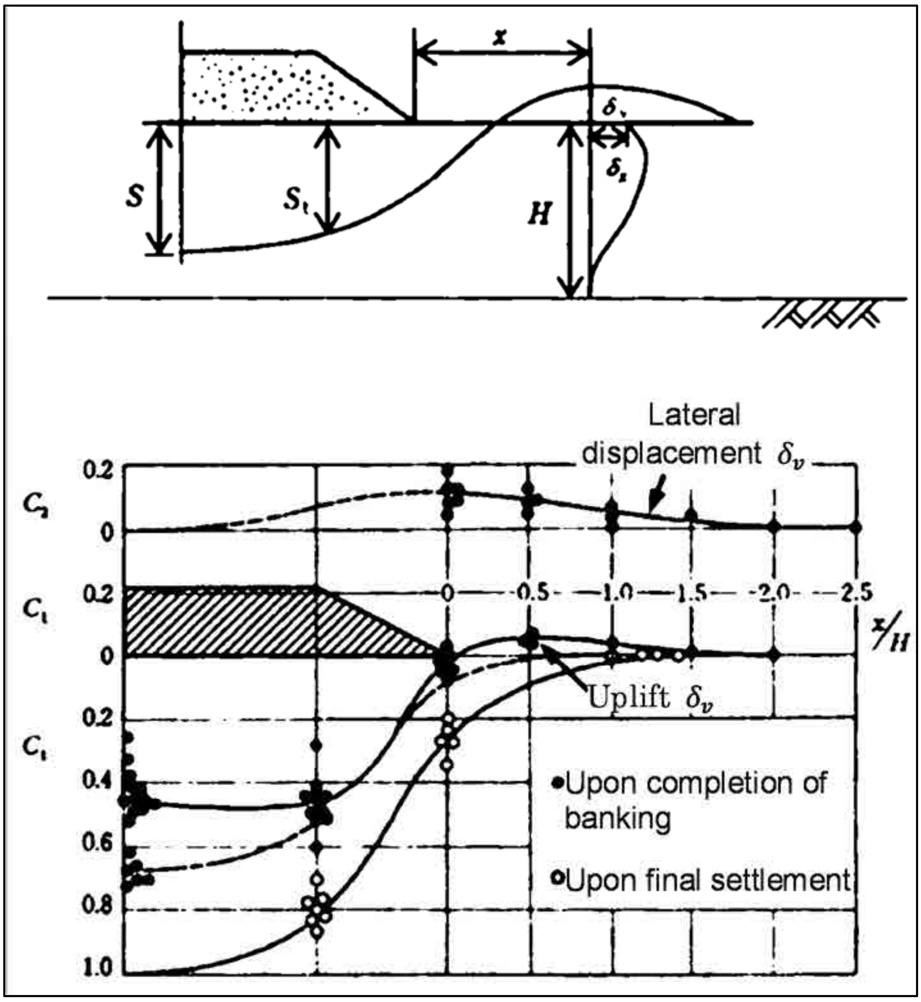


Figure 3-36 Settlement shape of embankment and influence on the lateral side

(2) Influence on the surrounding ground upon failure of the embankment

Some embankments constructed on soft ground for expressways and national highways, including test embankments, have collapsed or shown shape changes close to failure, and their dynamic states were mostly clarified by detailed observation. The degree and range of influence of these shape changes on the surrounding ground for these embankments are shown in Figure 3-37. This indicates that the range of influence greatly expands as the ratio of the embankment base width to the soft layer thickness and the soft layer thickness increase. In particular, when the foundation of the soft layer is slanted, it is expected that a failure like a landslide in the direction of the slant would occur and affect an extremely wide area, and thus sufficient caution is required.

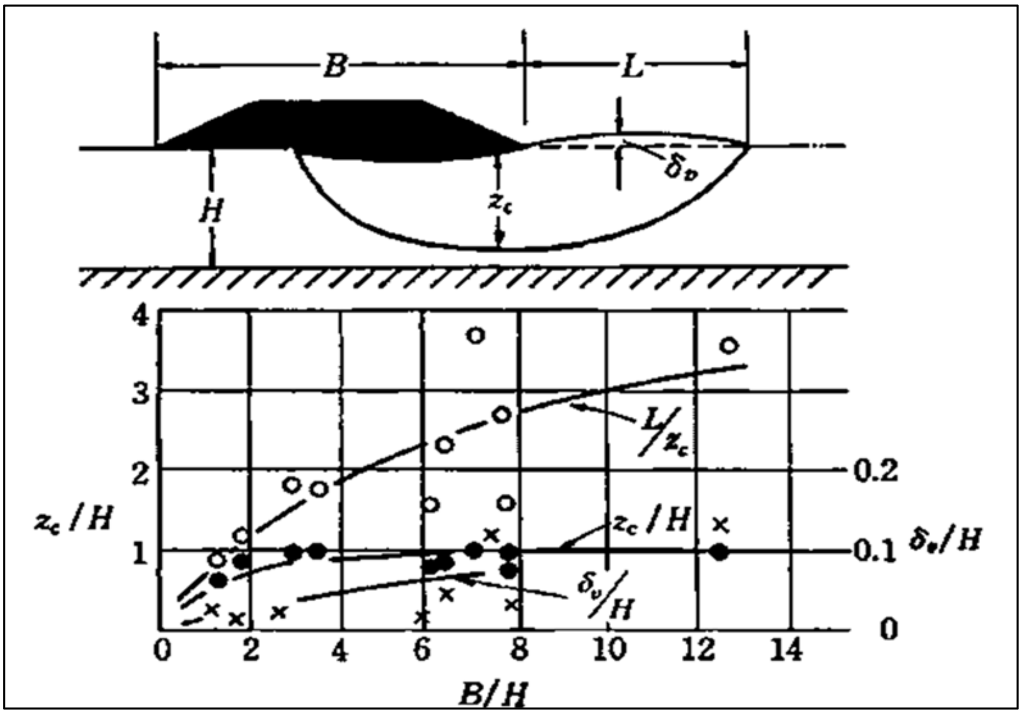
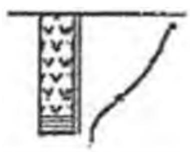
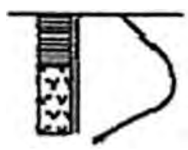
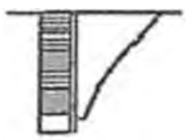
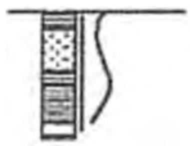


Figure 3-37 Influence of failure on surrounding ground

(3) Analysis method of lateral deformation with a complicated soil composition

NEXCO categorizes types of ground that have a relatively complicated soil composition and are not provided with any improvement into four types, as shown in Table 3-11, so as to identify problems with lateral deformation.

Table 3-11 Classification of ground deformations

Item \ Classification	Ground type 1	Ground type 2	Ground type 3	Ground type 4
Soil composition	Organic soil + cohesive soil	Cohesive soil + organic soil	Mainly cohesive soil	Alternation of sand and cohesive layer
Schematic diagram of subsurface displacement				
Characteristics	Large deformation occurs in organic soil layer down from the ground surface.	Subsurface deformation is affected by a organic soil layer composed of cohesive soil and organic soil down from the ground surface.	Subsurface deformation is due to a cohesive soil layer down from the ground surface.	Subsurface deformation is affected by a cohesive soil layer composed of an alternation of sand and cohesive soil.

Ground conditions used in this procedure are the soft layer thickness H shown in Figure 3-38 and the average undrained shear strength $\bar{c}$ from Eq. (3-32). For Type 1 ground, however, the thickness of the organic soil layer and undrained shear strength are used.

$$\bar{c} = \sum (c_i \cdot H_i) \,/\, \sum H_i \quad \text{Eq. (3-32)}$$

The embankment conditions used are the embankment height h_E shown in Figure 3-39 and the base width B_l from the embankment center (including the counterweight fill).

The amount of lateral deformation is analyzed with following three items.

a. Influence area L_1: Area from the embankment slope toe where the vertical displacement is over ± 5 cm

b. Vertical displacement δ_v: Maximum value of vertical displacement of the ground surface displacement stake (cm)

c. Lateral displacement δ_H: Maximum value of lateral displacement of the ground surface displacement stake (cm)

The results of the arrangement based on these conditions are shown in Figure 3-40.

<For Type 1 ground>

$$N(P) = \frac{\gamma_E \cdot h_E}{\bar{c}(P)} \quad \text{Eq. (3-33)}$$

<For Type 2 to 4 ground>

$$N = \frac{\gamma_E \cdot h_E}{\bar{c}} \quad \text{Eq. (3-34)}$$

(P) indicates the presence of organic soil layer.

$N(P), N$ *: Stability factor*

$\bar{c}(P)$ *: Average undrained shear strength of organic soil layer (kN/m²)*

γ_E *: Unit weight of embankment (kN/m³)*

h_E *: Embankment height (m)*

$\bar{c}$ *: Average undrained shear strength of soft layer (kN/m²)*

$H(P)$ *: Thickness of organic soil layer (m)*

H *: Soft layer thickness (m)*

B_1 *: Base width from the embankment center (m)*

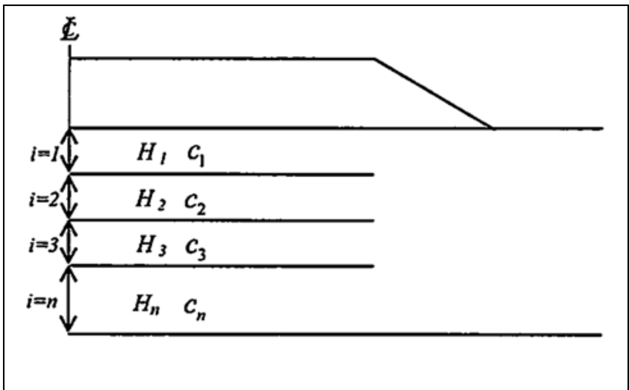


Figure 3-38 Thickness and undrained shear strength for each layer

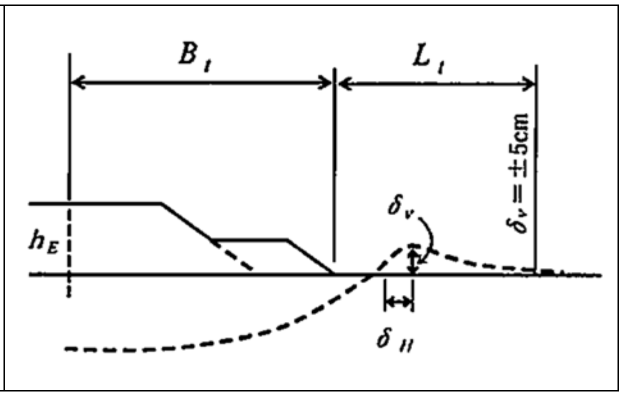


Figure 3-39 Definition of the amount of deformation and embankment dimensions

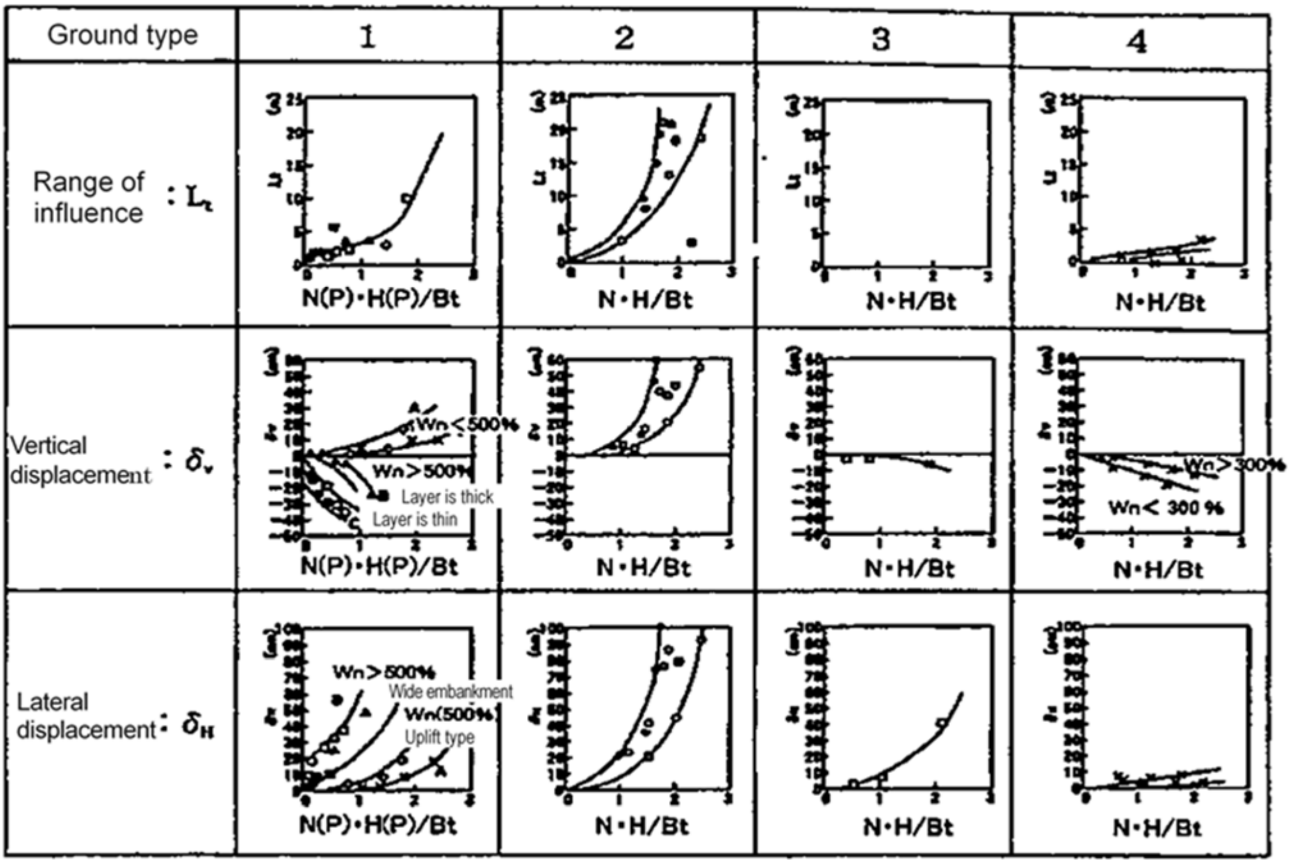


Figure 3-40 Lateral deformation and parameters

3-6-4 Finite Element Method

Combining the above-mentioned methods based on actual measurements, recently deformation of the surrounding ground is often predicted using the finite element method (FEM). Particularly when an embankment is irregularly shaped, (e.g., with embankment widening) and the ground is sloped in a way in which the above-mentioned measurement conditions do not apply, or when the embankment is treated with improvement work, it is necessary to confirm if the structure satisfies the necessary tolerances. In these cases, no methods other than FEM will be able to meet the purpose. FEM is also used in predicting one-dimensional consolidation. It should be noted, however, that the contents of an analysis or the its results will greatly vary depending on the method of analysis or the input parameter settings. Therefore, it is extremely important to evaluate the validity of the study results using FEM by referring to cases of similar work or trial embankment filling results.

The most important element in solutions to static loads produced by FEM is the ground composition model used in the analysis. Some of the models often used for soft ground are explained below. Popular composition models and their characteristics are compiled in Table 3-12. Although various composition models that better express soil materials showing complicated behavior have been proposed, models capable of expressing complicated behavior tend to require a large number of input parameters, or do not allow rational determination of solutions with general soil investigation or soil tests.

Table 3-12 Commonly used composition models and their characteristics

	Linear elasticity	Duncan-Chang	Elastic total plasticity (Mohr-Coulomb)	Correction Cam-Clay	Sekiguchi–Ota
Number of input parameters	Small	Medium	Medium	Large	Large
Calculation cost	Low	Medium	Medium	High	High
Stress-strain relationship	Linear	Hyperbolic curve (nonlinear elasticity)	Elastic region or linear	Nonlinear (elastoplasticity)	Nonlinear (elastoplasticity)
Expression of load removal and re-loading	–	++	+	++	++
Dependence of elastic modulus on confining pressure	-	++	-	++	++
Failure behavior	-	+	++	++	++
Dilatancy	-	-	++	+	+
Consolidation analysis	-	-	-	++	++
Anisotropy	-	-	-	-	++
Creep behavior	-	-	-	-	++
Influence of difference in initial stress or analysis step on final result	None	Exists	Exists	Exists	Exists

++ : Suitable, + : Suitable depending on the case, - : Not suitable

Since the precision of deformation analysis is determined by the combined effects of what composition model is used and how the input parameters are determined, the use of a complicated composition equation does not necessarily improve prediction precision. When selecting the composition model and setting input parameters, it is therefore necessary to study carefully the analysis target, select the appropriate composition model, and properly set input parameters taking into account the available ground data, expected stress levels, and strain levels.

Particularly when a nonlinear model for elastoplasticity is used, the initial stress state will have a great influence on the analysis results. There are a number of methods for analyzing the initial stress state, including a method using a program for dead load analysis or a method with input data setting, and they require sufficient examination. Moreover, there are cases in which the calculation results of some non-linear programs do not converge or produce an unnatural deformation or stress state. Therefore, it is necessary to sufficiently study the results. It is also necessary to sufficiently examine the final analysis results by comparing them with the expected characteristics of the phenomenon, similar analysis results, or past construction project data.

3-7 Critical Embankment Height and Filling Rate

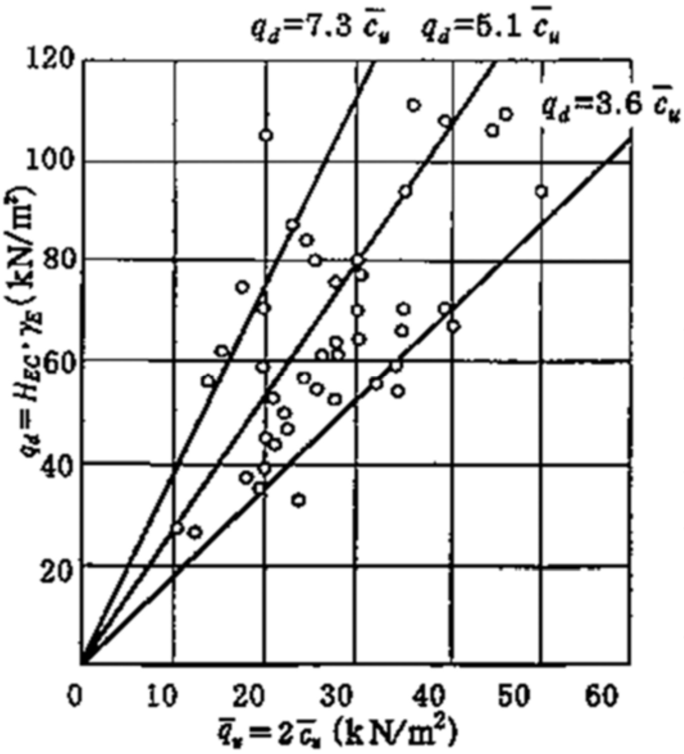
3-7-1 Critical Embankment Height

The critical embankment height is usually calculated from the unconfined compressive strength of the ground with a method using stability calculation or Taylor's Slope Stability Charts. When the limit of the bearing capacity of the ground q_d relative to the average unconfined compressive strength $\bar{q}_u$ of the soft soil layer calculated from Figures 3-8 and 3-9 is calculated from the relationship shown in Figure 3-41, the critical embankment height H_{EC} can be calculated from Eq. (3-35).

$$H_{EC} = q_d/\gamma_E \quad \text{Eq. (3-35)}$$

H_{EC} *: Critical embankment height (m)*
q_d *: Ultimate bearing capacity of ground (kN/m²)*
γ_E *: Unit weight of banking material (kN/m³)*

Figure 3-42 shows the relationship between the average water content ratio $\bar{w}_n$ of soft ground encountered in road construction for the Meishin Expressway, Tomei Expressway, etc., and the height of the embankments built on this ground H_E. When the critical embankment height H_{EC} calculated from the relationship between $\bar{w}_n$ and $\bar{q}_u$ in Figure 3-9 and the relationship between q_d and $\bar{c}_u$ in Figure 3-41 (assuming the unit weight of the banking material is 17 kN/m³) is put into the figure, we can see that the embankments were built up to a height almost equal to the critical height at the majority of soft ground sites. As shown by these examples, for most soft ground sites, there are very few cases of embankment failure with no special improvement (note that the filling rate was less than 3 to 10 cm per day).



Ground conditions	q_d (kN/m2)
Thick cohesive soil ground and muck, or peaty ground with thick deposit of organic soil	$3.6\,\overline{Cu}$
Ordinary cohesive soil	$5.1\,\overline{Cu}$
Thin cohesive soil ground and muck, or thin peaty ground with almost no organic soil inter-bedded	$7.3\,\overline{Cu}$

Figure 3-41 Relationship between average qu and q_d

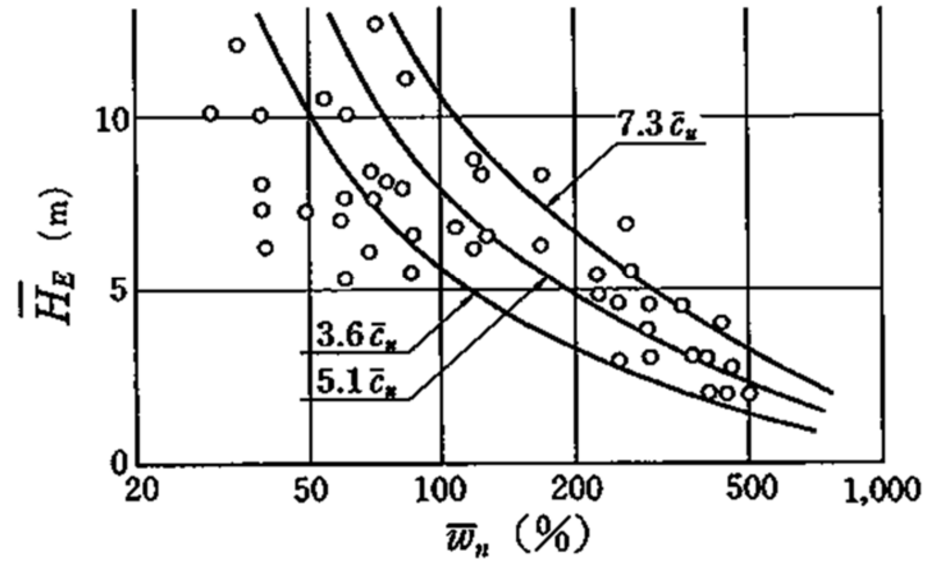


Figure 3-42 Average water content ratio $\overline{w}_n$ and embankment height H_E

3-7-2 Filling Rate

Since the filling rate has a large influence in a detailed study of consolidation settlement, it is determined prior to the settlement calculation. Table 3-13 shows filling rate values for which it is known according to past experience that ground failure does not occur. These values may be used as a reference in determining the filling rate, taking into account the ground conditions and embankment work based on the project plan or work plan. The values calculated as per this description do not always need to be adopted if the embankment is evaluated for its stability during works.

Figure 3-43 shows the relationship between total settlement at the center of the embankment in Figure 5-34 and the safety factor of the embankment against slides., The timing of t_{01} can be estimated in Figure 3-43(a) when the embankment height becomes H_E under the embankment speed of v_1, and the center settlement of the embankment becomes S_{t01} . If we interrupted embankment at this timing, consolidation would proceed gradually to stable status and primary consolidation would finish with the final settlement of S_{t1}. However, if we continued embankment after t_{01}, the settlement would increase rapidly as shown by the dotted line, and slip failure would occur on reaching the critical height of H_{Ec1}. Figure 3-43(b) shows the above relationship between time and safety factors. If the embankment speed is V_2, which is smaller than V_1, it will take more time to complete embankment. During the period until the completion of embankment, primary consolidation of the ground progresses more than in the case of V_1. Because the ground strength increases, embankment to a critical height of H_{Ec2} is possible, which is higher than the critical embankment height H_{Ec1} at a speed of V_1. Therefore, for safer and higher embankment, it is necessary to conduct embankment at a slow speed appropriate for consolidation of the ground, as shown in Figure 3-43.

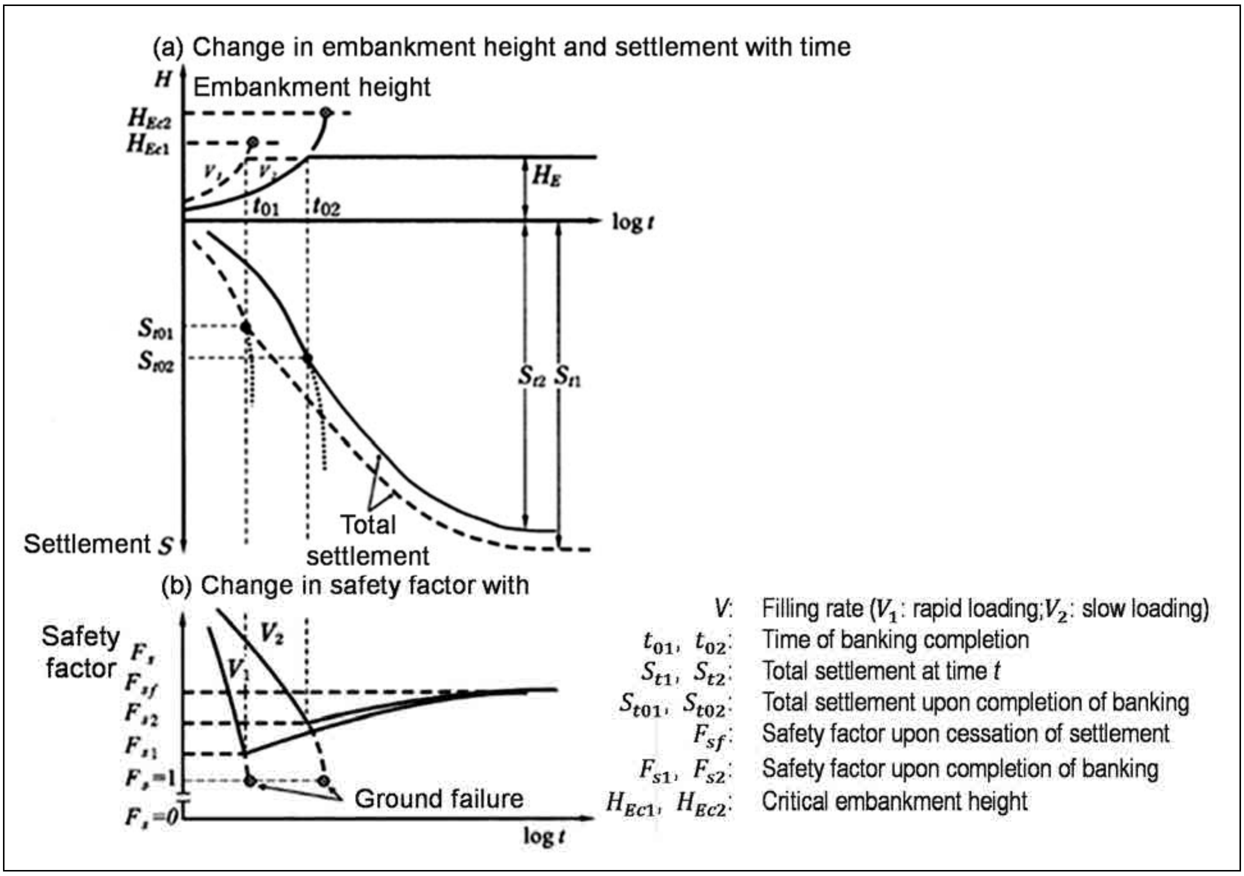


Figure 3-43 relation between banking speed and settlement, safety

The filling rates given in Table 3-13 may be used to calculate the work period in the preliminary study stage according to the nature of the ground. When an embankment is built at a high filling rate because of a limited work period, it is necessary to perform a detailed soil investigation and ensure the stability of the embankment work.

Table 3-13 Embankment filling rate

Ground conditions	Filling rate (cm/day)
Thick cohesive soil ground and muck, or peaty ground with thick deposit of organic soil	3
Ordinary cohesive ground	5
Thin cohesive soil ground and muck, or thin peaty ground with almost no organic soil inter-bedded	10

3-8 Lateral Movement of Retaining Walls

For soft ground, methods for studying lateral movement of the retaining wall of a pile foundation include the method specified by the Specifications for Highway Bridges intended for abutments, a structure of similar construction, and the simplified decision equation used by NEXCO.

(1) Prediction of Lateral Movement

The following is a simplified equation specified in the “Specifications for Highway Bridges, Part 4: Substructures”. If the lateral movement rate I is 1.2 or more in the case of Figure 3-44, the lateral movement is expected.

$$I = \mu_1 \cdot \mu_2 \cdot \mu_3 \cdot \frac{\gamma \cdot h}{c} \quad \text{Eq. (3-36)}$$

μ_1 : *Correction factor related to soft layer thickness (= H/L)*
μ_2 : *Correction factor related to foundation body resistance width (= b/B)*
μ_3 : *Correction factor related to the length of the abutment (= H/A (≦3.0))*
γ : *Unit weight of embankment (kN/m^3)*
h : *Embankment height (m)*
c : *Average cohesion of soft layer (kN/m^2)*
H : *Soft layer thickness (m)*
A : *Abutment length (m)*
B : *Abutment width (m)*
b : *Sum of the widths of the foundation body (m)*
L : *Foundation penetration depth (m)*

The following lateral flow index F value is used in NEXCO for the prediction of lateral movement. When F is not less than $4.0 \times 10^{-2} m^{-1}$, there is no sign of lateral movement as shown in Figure 3-45.

$$F = \frac{c}{\gamma \cdot h} \cdot \frac{1}{H} \quad \text{Eq. (3-37)}$$

c : *Average cohesion of soft layer (kN/m^2)*
γ : *Unit weight of embankment (kN/m^3)*
h : *Embankment height (m)*
H : *Soft layer thickness (m)*

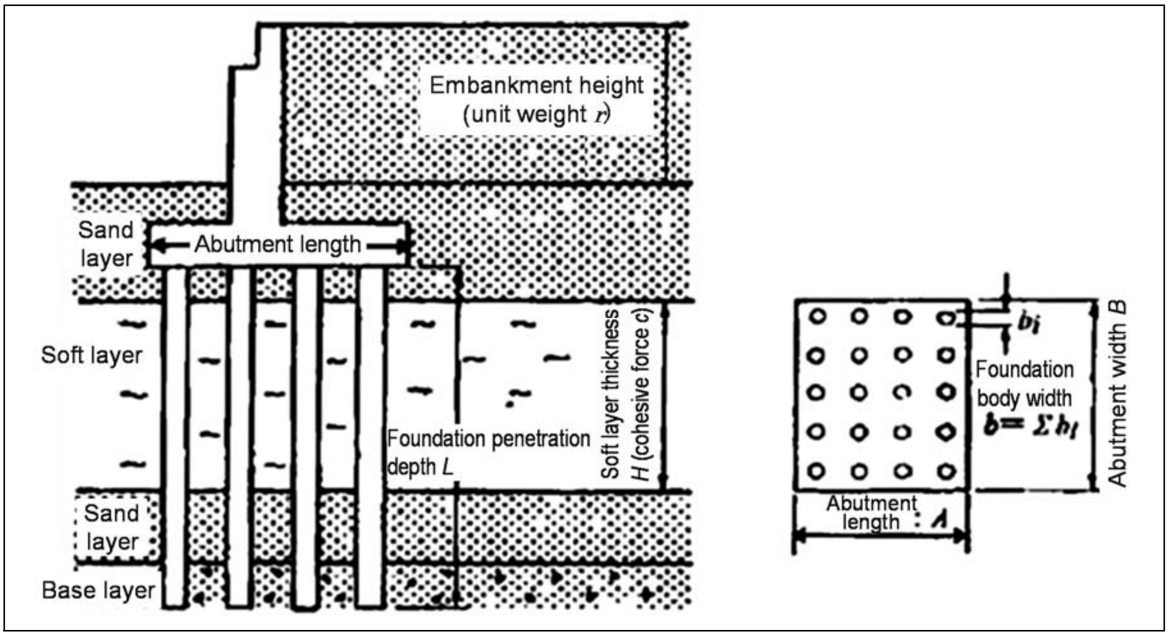


Figure 3-44 Foundation under eccentric load

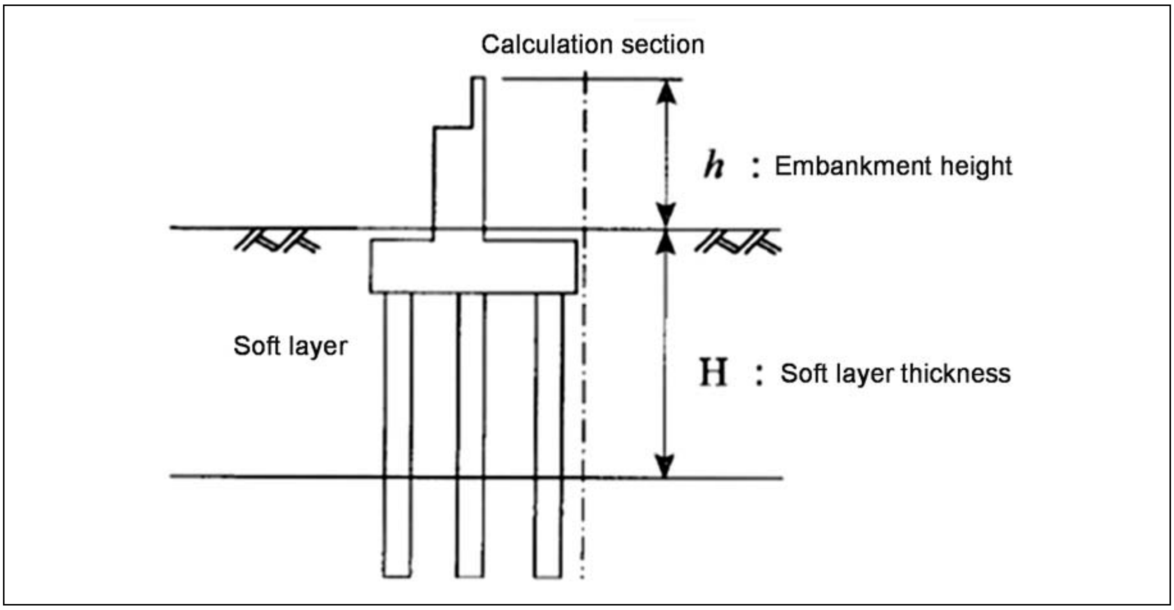


Figure 3-45 Explanatory diagram of the calculation method of lateral flow index *F*

(2) Amount of Lateral Movement

One of the empirical estimating methods of amount of lateral movement (δ) of abutments is introduced by NEXCO. if $F_R \geqq 3$ and $\delta \leqq 10$ cm in the following equivalent, the impact of lateral movement may be small.

$$\delta = \beta \cdot \varepsilon \cdot H \quad \text{Eq. (3-38)}$$

β : *Correction factor (taken as 0.5 on an empirical basis)*

ε : *Soil strain, which is obtained from the following equation:*

$\varepsilon = -0.72 \cdot (q_u/E_{50}) \cdot 1n(1 - 1 / F_R)$

H : *Thickness of a soft cohesive soil layer in the soft layer (m)*

The relationship between F_R and displacement is shown in Figure 3-46.

q_u : Unconfined compressive strength (kN/m^2)

E_{50} : elastic modulus obtained from unconfined compression test (kN/m^2)

F_R : Safety factor of the ground against failure due to embankment loads, which is shown by Eq. (3-39).

$$F_R = \frac{\alpha_1 \cdot c + \alpha_2 \cdot c_A \,/\, H + \alpha_3 \cdot (1\,/\,2) \cdot \left(\gamma_1 \cdot H' \cdot N_\gamma\right)}{\gamma_t h} \quad \text{Eq. (3-39)}$$

α_1 : *Correction factor related to the hypothesized slip surface shape (4 for pile and 2 for caisson)*

α_2 : *Correction factor due to increase in cohesion or presence of foundation structure in the ground near the underside of the soft layer (5 for pile and 2.5 for caisson)*

α_3 : *Correction factor related to the compaction effect of sand ground due to embankment loads (3 for pile and 0 for caisson)*

c : *Average cohesion of soft layer (kN/m^2)*

A : *Length of abutment in the bridge axis direction (m)*

γ_1 : *Unit weight of sand layer (kN/m^3)*

H' : *Thickness of sand layer in soft ground (m)*

h : *Embankment height (m)*

H : *Thickness of soft cohesive soil layer in soft layer (m)*

N_γ : *Bearing capacity factor of the ground*

γ_t : *Unit weight of embankment (kN/m^3)*

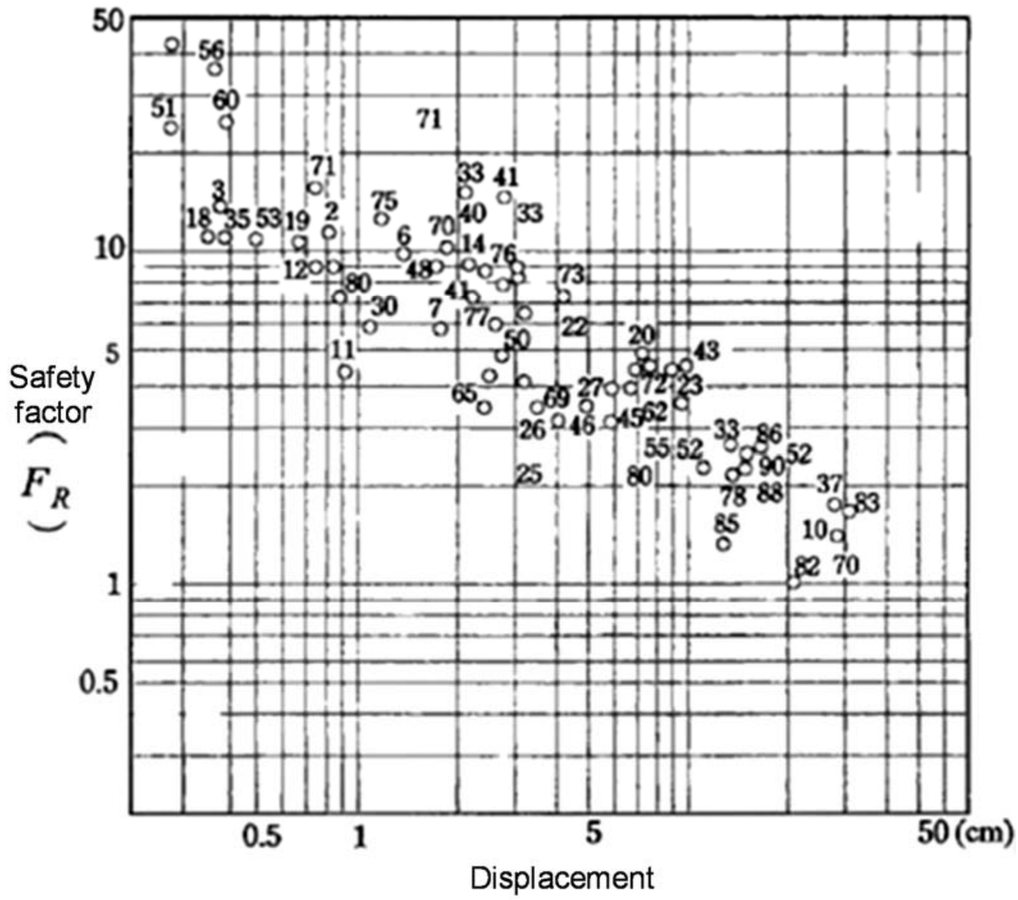


Figure 3-46 Relationship between F_R and displacement

3-9 Stability against Seismic Ground Motion

Earthquake-induced damage to an earthwork structure on soft ground is generally divided into two types, one attributable to the liquefaction of a saturated loose sandy soil layer, and the other attributable to the softening of a particularly soft cohesive soil layer as a result of the application of cyclic shear stress.

3-9-1 Liquefaction

The damage to an earthwork structure due to liquefaction of the foundation ground becomes large according to the shallowness and thickness of the layer of loose sandy soil. It is rare for the cohesive soil to cause serious damage due to the actions of seismic motion, because the strength increases due to the load of the earthwork structure with the passage of time of the consolidation process after construction (primary and secondary consolidation). Therefore, general checking against the actions of seismic motion is not necessary on cohesive soft ground, on the condition that appropriate treatment of the foundation ground, careful compaction, and installation of drainage works were properly conducted. However, embankments are deformed due to liquefaction of embankment materials lower than the groundwater level at locations where the groundwater level is by consolidation settlement.

Checking against the actions of ground motion is necessary for embankments where major damage is anticipated, such as;

a. road sections that cannot be easily repaired if they suffer earthquake damage,

b. first-importance embankments that might cause secondary damage to facilities in the vicinity if they were damaged by an earthquake,

c. embankments on thick depositions of loose sandy soil layers near former river channels, reclaimed land, or waterfront areas where large damage would be likely to occur due to liquefaction,

d. high embankments with particular depositions of soft cohesive soil, etc.

(1) Factors of Liquefaction

When ground with sandy soil that supports an embankment is liquefied during an earthquake, the bearing capacity of the ground will be seriously lost, which can eventually cause major damage to the embankment. When the site is evaluated to have ground with such conditions, it is recommendable to perform a study with consideration of the impact of liquefaction.

Liquefaction occurs in sandy soil when a negative dilatancy resulting from the sandy soil being subjected to vibration is converted to pore water pressure, the effective stress of the ground is reduced, and the strength or bearing capacity of the ground is eventually totally or partially lost. Whether or not ground with sandy soil will liquefy in a major earthquake is usually dominated by the following factors.

Ground motion

The greater the ground motion, the greater the shear stress in the ground in an earthquake, and the possibility of liquefaction will become higher. On the other hand, depending on the conditions of the ground, no liquefaction will occur unless there is ground motion strong enough to cause shear stress during an earthquake beyond the critical level. The longer the duration of the ground motion, the higher the possibility of liquefaction, because the number of wavelengths effective in causing liquefaction increases.

Density and confining pressure

It becomes harder for liquefaction to occur as the density of the ground increases. When the type of soil and the confining pressure are constant, the N-value will be higher as the soil density increases. Therefore, the greater the N-value, the harder it is for liquefaction to occur. For a constant density, because the N-value increases as the confining pressure increases, evaluation of liquefaction is carried out using a modified N-value converted to an N-value corresponding to a constant confining pressure. In the case of a soil layer composed of coarse sand containing gravel with coarse particles, the N-value measured for the actual dynamic shear strength ratio will be larger, which requires attention. On the other hand, a smaller N-value will be measured in fine sand ground containing silt and other fine particles.

Grain size distribution

The grain size distribution of sandy soil also affects liquefaction. This is because when sandy soil contains a large volume of fine-grained constituents, particularly clay constituents, the degree of strength degradation of the particles will be reduced, as these soil particles tend to be unlikely to become loose even though the pore water pressure increases. On the other hand, when the grain size is large, the excess pore water pressure that occurs in the ground is easily dissipated, which decreases the degree of effective stress reduction. Figure 3-47 is an example of the range of grain sizes for ground where liquefaction actually occurred or where it is known that liquefaction is likely.

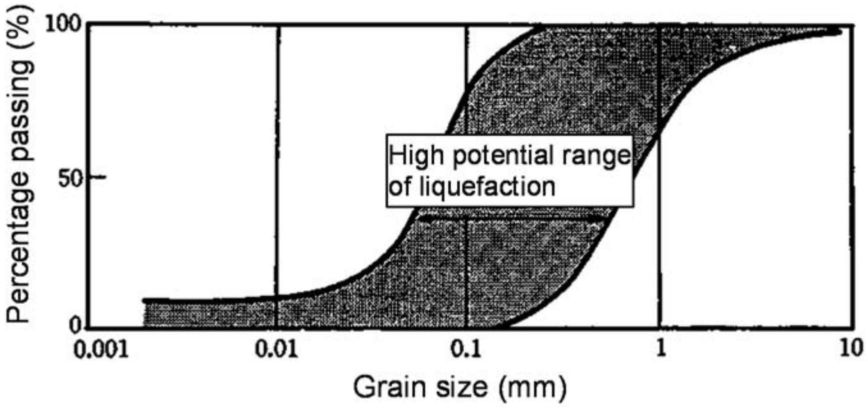


Figure 3-47 Grain size distribution with high potential of liquefaction

4) Vertical load, groundwater level, and organization of strata

Dynamic shear strength and shear stress that occur in an earthquake increase as the depth in the ground increases. However, in general, liquefaction tends to become more unlikely as the depth increases, as a vertical load such as an embankment increases, or as the groundwater level decreases. This is because the dynamic shear strength increases in proportion to an increase in the effective overburden pressure, while the shear stress that occurs in an earthquake does not increase in linear proportion to an increase in the effective overburden pressure. The aspects of liquefaction also change remarkably depending on the soil composition. For example, if there is a soil layer in the subsurface of the ground which is three meters or more in thickness and resistant to liquefaction, sand and water boiling, which are signs of the liquefaction phenomenon, are not observed even if liquefaction occurs under the ground. Therefore, it can be assumed that liquefaction deeper in the ground has less affect on a structure than liquefaction in the shallow part of the ground.

(2) Potential Ground for Liquefaction

There are a variety of study and evaluation methods to evaluate the possibility of ground liquefaction according to each stage of study. In an initial investigation stage or a preliminary investigation stage when there is a small amount of detailed information on the ground or soil properties, a rough understanding of the ground is determined with respect to the possibility of liquefaction in an earthquake based on available information such as the details of the topography or N-value. The subsequent investigation planning or study methods are then evaluated.

1) Rough determination based on topography, geology

Based on past earthquake experience, it is known that that liquefaction generally occurs in alluvial ground or artificially reclaimed ground. In particular, liquefaction is more likely to occur at a location near a former river channel, a reclaimed land site, or a waterfront area with depositions of saturated loose sandy soil. Table 3-14 tabulates liquefaction potential with a focus on topographic characteristics.

Table 3-14 Liquefaction potential based on topography

Detailed topographical classification	Liquefaction potential
Reclaimed land, embankment on water, present or former river channel, only slightly developed natural dam, lowland between hills, boundary between hills and lowland	Highly possible
Lowlands other than above	Possible depending on the case
Plateau, hill, mountain	Small possibility

2) Rough determination based on the N-value or grain size study results

While quantitative evaluation methods based on the N-value or grain size test results, N-values below 10–15 as shown in Table 1-3 may be used in an initial or preliminary study stage as a guideline for the range of N-values likely to suffer damage due to liquefaction.

3) Potential soil layers for liquefaction

Since a sandy soil layer located in an alluvial stratum is likely to cause liquefaction that will affect earthwork structures in the event of a major earthquake if all of the following three conditions are met, liquefaction evaluation of that layer is necessary.

a. The groundwater level is within 10 m from the ground surface, and a saturated soil layer is located at a depth less than 20 m from the ground surface

b. A soil layer in which the fine constituent content F_c is 35%, or the plasticity index I_p is less than 15 even if F_c is over 35%

c. The median grain size D_{50} of a soil layer is less than 10 mm, and the 10% grain size D_{10} is less than 1 mm

(3) Liquefaction Analysis

Regarding soil layers for which it is necessary to evaluate liquefaction as per (i) above, the resistance factor against liquefaction F_L is calculated from Eq. (3-40), and a soil layer that has a value of 1.0 or less is considered to be ground that could suffer liquefaction.

$$F_L = R / L \quad \cdots\cdots \text{Eq. (3-40)}$$

$$R = C_w \cdot R_L \quad \cdots\cdots \text{Eq. (3-41)}$$

$$L = \gamma_d \cdot k_h \cdot (\sigma_v / \sigma_v') \quad \cdots\cdots \text{Eq. (3-42)}$$

$$\gamma_d = 1.0 - 0.015x \quad \cdots\cdots \text{Eq. (3-43)}$$

<for Level 1 Ground Motion and Type I of Level 2 Ground Motion>

$$C_w = 1.0 \quad \cdots\cdots \text{Eq. (3-44)}$$

<for Type II of Level 2 Ground Motion>

$$\begin{aligned} C_w &= 1.0 && (R_L \leqq 0.1) \\ C_w &= 3.3 \cdot R_L + 0.67 && (0.1 < R_L \leqq 0.4) \quad \cdots\cdots \text{Eq. (3-45)} \\ C_w &= 2.0 && (0.4 < R_L) \end{aligned}$$

F_L : *Resistance factor to liquefaction*
R : *Dynamic shear strength ratio*
L : *Seismic shear stress ratio*
C_w : *Correction coefficient for seismic motion characteristics*
R_L : *Cyclic triaxial strength ratio, which is to be determined by the provisions of (iii)*
r_d : *Reduction factor for seismic shear stress ratio in the depth direction*
k_h : *Design horizontal seismic coefficient for liquefaction evaluation specified by Table 3-6*
σ_v : *Total overburden pressure at a depth x (m) from the ground surface (kN/m²)*
σ_v' : *Effective overburden pressure at a depth x (m) from the ground surface (kN/m²)*
x : *Depth from the ground surface (m)*

Cyclic triaxial strength ratio

The cyclic triaxial strength ratio R_L is calculated from Eq. (3-45).

$$(N_a < 14)\ :\ R_L = 0.0882 \cdot \sqrt{N_a/1.7} \quad \cdots\cdots \text{Eq. (3-46a)}$$

$$(N_a \leq N_a)\ :\ R_L = 0.0882 \cdot \sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} \quad \cdots\cdots \text{Eq. (3-46b)}$$

<Sandy soil>

$$N_a = c_1 \cdot N_1 + c_2 \quad \cdots\cdots \text{Eq. (3-47)}$$

$$N_1 = 170 \cdot N/(\sigma'_{vb} + 70) \quad \cdots\cdots \text{Eq. (3-48)}$$

$$c_1 = \begin{cases} 1 & (0\% \leqq F_c < 10\%) \\ (F_c + 40\ /\ 50) & (10\% \leqq F_c < 60\%) \\ F_c/20 - 1 & (60\% \leqq F_c) \end{cases} \quad \cdots\cdots \text{Eq. (3-49)}$$

$$c_2 = \begin{cases} 0 & (0\% \leqq F_c < 10\%) \\ (F_c - 10)\ /\ 18 & (10\% \leqq F_c) \end{cases} \quad \cdots\cdots \text{Eq. (3-50)}$$

<Gravelly soil>

$$N_a = [1 - 0.36 \cdot \log_{10}(D_{50}\ /\ 2)]N_1 \quad \cdots\cdots \text{Eq. (3-51)}$$

R_L : *Cyclic triaxial strength ratio*
N : *N-value obtained from the standard penetration test*
N_1 : *N-value converted as equivalent to effective overburden pressure of 100 kN/m^2*
N_a : *Corrected N-value taking into account the effect of grading*
σ'_{vb} : *Effective overburden pressure at a depth from the ground surface when the standard penetration test was conducted (kN/m^2)*
c_1, c_2 : *Correction coefficient of N-value according to fine grained constituent content (kN/m^2)*
F_C : *Fine grained constituent content (%) (passing percentage by mass of soil particles not more than 75 μm in grain size)*
D_{50} : *50% grain size (mm)*

Design horizontal seismic coefficient

To calculate the design horizontal seismic coefficient k_h used for ground liquefaction evaluation, values calculated from Eq. (3-51) may be used based on the standard values of design horizontal seismic coefficients shown in Table 3-14. For calculating local correction factors and ground classification for seismic design.

$$k_h = c_z \cdot k_{h0} \quad \cdots\cdots \text{Eq. (3-52)}$$

k_h : *Design horizontal seismic coefficient (rounded to two decimal places)*
k_{h0} : *Standard value of design horizontal seismic coefficient based on Table 3-15*
c_z : *Local correction factor*

Table 3-15 Standard design values of horizontal seismic coefficients for liquefaction evaluation

Seismic motion		Type of ground: Type I	Type II	Type III
Level 1 Seismic Motion		0.12	0.15	0.18
Level 2 Seismic Motion	Type I	0.30	0.35	0.40
	Type II	0.80	0.70	0.60

(4) Improvement of Soil Layers Subject to Liquefaction

As explained above, for sandy soil layers where it is evaluated that liquefaction could occur, the impact of liquefaction is appropriately treated according to the earthwork structure or soft ground improvement work by taking into account the occurrence of excess pore water pressure, reduction in shear strength, or reduction in rigidity. While various methods have been proposed to reduce soil parameters of sandy soil layers, it is necessary to select the right one depending on the checking method. When checking the foundation of an earthwork structure (e.g., a retaining wall), it is recommendable to reduce the relevant soil parameters according to the resistance to liquefaction factor F_L, the depth from the original ground surface, and the value of dynamic shear strength ratio R.

3-9-2 Stability Analysis in the Event of Earthquake

The basic process of stability of an earthwork structure on soft ground against the actions of seismic motion is to confirm that

a. no failure (e.g., ground slip failure) will occur despite the actions of seismic motion working on the structure while it is open to services

b. no settlement will occur that would have a negative impact on the functions of the earthwork structure (e.g., the trafficability of road surface)

c. no harmful deformation (e.g., settlement, uplift, or lateral displacement) will occur in facilities or ground in the vicinity.

The analysis techniques described below are relatively widely used methods that take into account applicability to practical operations. Conventionally, stability analysis is conducted using methods that check the safety factor according to stability calculations with, for example, a seismic coefficient method that assumes a circular slip surface in checking seismic resistance. Stability analysis techniques are designed to check whether or not a structure is stable, and are not designed to directly assess residual deformation of the structure. However, these techniques can be used on an empirical basis to evaluate a structure’s deformation performance or damage severity using a safety factor based on analysis results for past cases of damage. A number of stability analysis techniques are explained below, based on the knowledge accumulated thus far or the current state of technology. When the ground conditions are complicated or particularly important structures need to be checked for their seismic resistance, it is recommendable to use residual deformation analysis to ensure that the amount of residual settlement in the event of a major earthquake can meet the allowable limit.

(1) Circular Slip Stability Analysis with Inertial Force

Although embankments on soft ground rarely collapse due to inertial force, if an embankment is expected to fail mainly due to inertial force, the safety factor can be calculated using Eq. (3-53). This equation uses regular strength, although the strength of soft is ground often reduced under the actions of seismic motion. Considering this, this equation is rationally applicable to embankments in mountain areas with no fear of serious strength reduction, or to embankments on flat land dominated by cohesive soil.

$$F_s = \frac{\sum\{c \cdot l + [(W - u_0 b) \cdot \cos\alpha - k_h \cdot W \cdot \sin\alpha] \cdot \tan\phi\}}{\sum(W \cdot \sin\alpha + (h / r) \cdot k_h \cdot W)} \quad \text{Eq. (3-53)}$$

F_s : Safety factor
c, ϕ : Cohesion of sand (kN/m^2) and angle of internal friction (°), respectively
W : Total weight of a slice (kN/m)
l : Length of the slip line of a slice (m)
b : Width of a slice (m)
u_o : Pore water pressure due to regular groundwater level (kN/m^2)
k_h : Design horizontal seismic coefficient
r : Radius of slip circle (m)
h : Vertical distance from the center of gravity of a slice to the center of the slip circle (m)
α : Angle formed by the tangent line on the slip line of a slice and the horizontal plane (°)

The design horizontal seismic coefficient k_h may be calculated from Eq. (3-54). For calculation of local correction factors and ground classifications for seismic design, see "Road Earthwork Guidelines: Actions of Seismic Motion, Data 1".

$$k_h = c_z \cdot k_{h0} \quad \text{Eq. (3-54)}$$

k_h : Design horizontal seismic coefficient (rounded to two decimal places)
k_{h0} : Standard value of design horizontal seismic coefficient, which is given in Table 3-16
c_z : Local correction factor

Table 3-16 Standard design values of horizontal seismic coefficient

Seismic motion		Type of ground Type I	Type II	Type III
Level 1 Seismic Motion	for inertial force	0.08	0.10	0.12
Level 2 Seismic Motion	for inertial force	0.16	0.20	0.24

The standard values for the design horizontal seismic coefficient given in Table 3-16 are established from the results of inverse analysis of cases of damaged and undamaged embankments affected by past earthquakes based on the assumption that stability calculation assuming a circular slip surface is used. When checking is conducted using methods other than this method, values from Table 3-16 are not used. See "Road Earthworks: Embankment Work Guidelines" for details.

In this technique, where thick cohesive soil ground is distributed as the foundation ground of an embankment, when inertial force equivalent to the design horizontal seismic coefficient is simply applied, in many cases the circular arc with the smallest safety factor is a deep circular arc that passes through the base on the cohesive soil ground. However, there are no cases of slip failure in deep circular arcs like these. Therefore, for the earthquake stability analysis of cohesive soil ground, it is recommendable to analyze stability in the event of a major earthquake for a circular arc that gives the smallest safety factor against normal actions.

Stability analysis techniques assuming a circular slip surface with inertial force taken into account include a method that uses total stress strength (dynamic strength) with the magnitude of cyclic shear strain taken to be the shear strength of soil, in addition to the procedure given as Eq. (3-53).

If the safety factor calculated from a slip stability analysis technique with inertial force taken into account turns out to be 1.0 or more, it is acceptable to believe that the amount of consolidation is sufficiently small or that the structure would be deformed only in a limited range.

(2) Circular Slip Stability Analysis with Excess Pore Water Pressure

This analysis technique estimates the reduction in shear strength of soil by increment of excess pore water pressure. The following method of stability analysis is a kind of simple analysis.

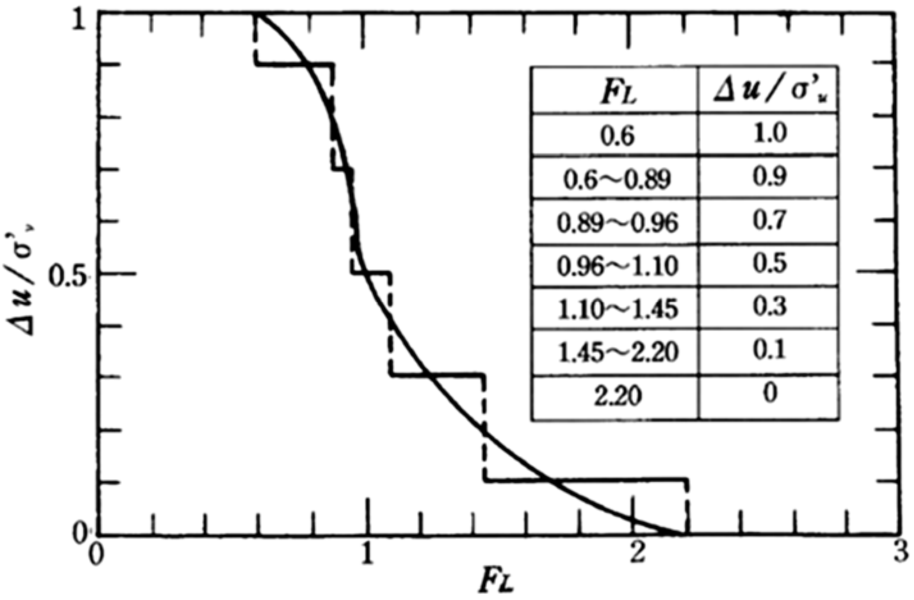
For stability analysis of an embankment on ground with the potential for liquefaction, there is a method that uses Eq. (3-55) to calculate the safety factor F_{sd}. This method evaluates the reduction in shear strength of the soil due to the actions of seismic motion according to the amount of increase in excess pore water pressure. It leaves out the effect of inertial force due to the actions of seismic motion, but conducts stability analysis taking into account excess pore water pressure Δ_u that occurs in saturated sandy soil ground during an earthquake.

$$F_{sd} = \frac{\sum(c \cdot l + (W - u_0 \cdot b - \Delta u \cdot b)\cos\alpha \cdot \tan\phi)}{\sum W \cdot \sin\alpha} \quad \text{Eq. (3-55)}$$

F_{sd} : *Safety factor*
$c,\ \phi$: *Cohesion of sand (kN/m²) and angle of internal friction (˚), respectively*
W : *Total weight of a slice (kN/m)*
l : *Length of the slip line of a slice (m)*
b : *Width of a slice (m)*
u_0 : *Pore water pressure due to regular groundwater level (kN/m²)*
Δ_u : *Excess pore water pressure induced by seismic motion (kN/m²)*
α : *Angle formed by the tangent line on the slip line of a slice and the horizontal plane (˚)*

Excess pore water pressure that occurs due to seismic motion may be calculated from Figure 3-48 by using the resistance factor against liquefaction F_L obtained from the results of liquefaction evaluation.

If the safety factor obtained from the stability analysis technique assuming a circular slip surface taking into account the occurrence of excess pore water pressure is 1.0 or more, it is acceptable to think that the amount of consolidation is sufficiently small or the structure would be deformed only to a limited extent. However, it has been clarified that this technique tends to produce safe-side calculation results. Therefore, this calculation method should be taken as the primary check, and it is recommendable to conduct analysis of seismic residual consolidation at the same time when the value is lower than the allowable safety factor.



F_L	$\Delta u/\sigma'_v$
0.6	1.0
0.6～0.89	0.9
0.89～0.96	0.7
0.96～1.10	0.5
1.10～1.45	0.3
1.45～2.20	0.1
2.20	0

Figure 3-48 Relationship between resistance factor against liquefaction F_L and excess pore water pressure ratio Δ_u/σ_v

3-9-3 Residual Deformation Analysis

A variety of methods ranging from simple to complicated have been proposed for seismic residual deformation analysis. In setting design target values or analysis method characteristics, it is important to adopt appropriate methods while taking into account the local conditions. When either method is used, it is very important to examine the input parameters and analysis results. This examination is an essential operation to maximize the precision of each analysis technique. The basic matters on FEM is described in "3-2-4 (4) Finite element method".

(1) Static Analysis Method Based on FEM

This is a static analysis method based on FEM that assumes ground deformation associated with liquefaction as caused by rigidity reduction of the liquefied layer. This method takes into account the influence of seismic motion only through the liquefaction evaluation without the action of inertial force. Post-quake rigidity of a liquefied layer can be obtained from Figure 3-49. When there is a layer evaluated as one in which liquefaction will not occur in the upper section of a layer where liquefaction could occur, it is necessary to appropriately take account of alteration of the properties of even the non-liquefaction soil layer. While various methods to reduce the soil parameters of a surface non-liquefaction layer have been proposed, methods that have proven consistent with actual measurements or experiment results should be used. It is recommendable to use a method or an elastoplastic model that reduces shear rigidity so that no tensile stress will occur in the surface non-liquefaction layer.

Since this is a two-dimensional FEM method, it is capable of solving problems of more complicated shapes than the stability analysis. It can also determine lateral deformation in addition to settlement on the top. Caution is necessary when this method is applied to unique seismic motion, as it evaluates the influence of seismic motion only with liquefaction evaluation and cannot directly evaluate the influence of frequency or continuation time.

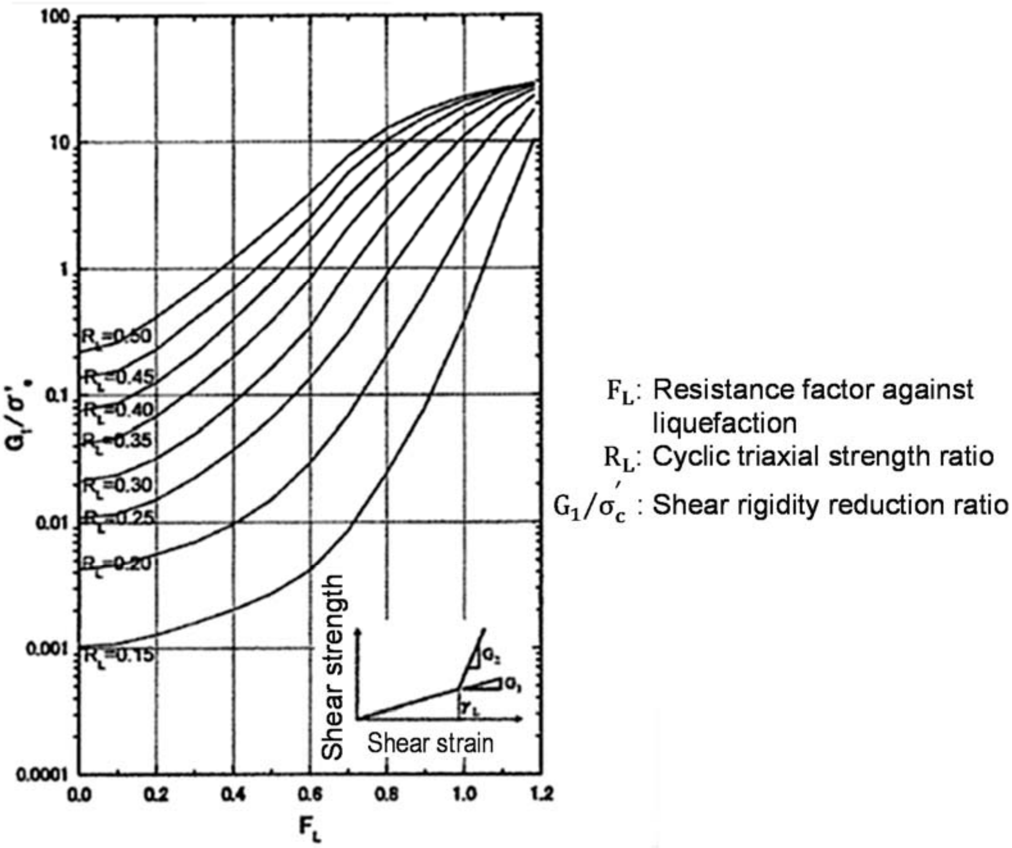


Figure 3-49 Reduction in shear rigidity of liquefaction prone layer

(2) Dynamic Effective Stress Analysis Method Based on FEM

This method is based on the most accurate modeling of actual phenomena (e.g., liquefaction) occurring due to an earthquake. For example, input the input seismic motion as the time history waveform of acceleration from the bottom of the section to be analyzed, and calculate the stress and displacement for each subdivided time step of the seismic motion time history. This method can take account of the occurrence of excess pore water pressure during an earthquake or the stress-strain relationship in liquefied soil.

These methods, however, generally require detailed soil investigation and have a large number of parameters that need to be set. They also have parameters not directly determined by test results. Therefore, the results can greatly vary depending on how the input parameters are set. In order to improve the reliability of the analysis results, it is very important to conduct detailed soil investigation according to the analysis technique, and to carefully examine the input parameters and analysis results, including confirmation of consistency with actual measurement results or experiment results and the parameter-setting method.

3-10 Experimental Construction and Investigation during Works

3-10-1 Experimental Construction

When an earthwork structure to be built on soft ground or soft ground improvement work is planned, designed, or constructed, experimental construction is carried out whenever it is evaluated to be necessary to check or verify the validity of the proposed improvement work, applied design method, designed soil properties or design values, and construction methods, or the applicability of new methods and new technologies. The period of experimental construction is broadly divided into the following items. The scale or method of the experimental construction will vary depending on the contents to be checked or the check items to be studied in the experimental construction.

(1) Experimental construction before main works

A trial to be carried out separately prior to commencing the main work is conducted for the purposes noted below.

Validation of soil properties and design values

In studying soft ground improvement work or earthwork structures to be built on soft ground, the ground information obtained by soil investigation is generally limited. In addition, since many hypotheses and uncertain factors are involved in analysis methods, there is no method that can provide a strictly accurate reproduction of the soft ground improvement work or the behavior of an earthwork structure to be built on soft ground. In experimental construction, a full-size earthwork structure is generally constructed. Using such a real structure, differences between the actual behavior and the theoretical predictions can be accurately verified to eventually achieve validation of the soil properties or design values. When the proposed site sits on large-scale soft ground and the soft ground improvement work is expected to have a major impact on the construction cost or period, or when the target value of residual settlement needs to be determined, it is recommendable to carry out experimental construction, understand the effect of the proposed soft ground improvement work, and validate the soil properties and design values. It is recommendable to carry out experimental construction on a scale as close to the main work as possible, analyze the trial results, and set a sufficient lead time that allows for sure incorporation of the analysis results in the main work. The behavior of the earthwork structure is measured, the analysis results and test results are compared, and the soil properties and design values are revised by means of reverse analysis. Depending on the degree of revision, the structure or scale of the earthwork structure is reviewed, or necessary corrective actions are taken, such as addition or modification of improvement work. Measurement items, methods, and locations with respect to experimental construction should be appropriately evaluated depending on the location for which the design values are to be verified or the surrounding conditions, with reference to “3-2-8(2) Investigation during Construction Stage”. These measurement items are shown in detail in “Chapter 5 Work Control and Maintenance”.

Confirmation of applicability of new technologies and methods

When a new technology or method is adopted, it is difficult to confirm the effect of the work or evaluate the validity of the construction machinery or material applicability only on the basis of a desk check. Therefore, it is recommendable to check these points based on full-size experimental construction.

2) Experimental construction before commencing or during construction

Trial construction conducted before commencing work or as needed during the execution of work is generally carried out where the design policy and construction plan have mostly been determined and it is necessary to test the applicability of the details. In such cases, experimental construction generally involves comparing materials or evaluating their quality, or simple trial checks of the applicability of the construction machinery or the construction method. It is carried out prior to commencing each type of work during the main work.

This type of experimental construction is also carried out when a condition different from the initial stage is encountered in the course of executing the work, and an execution method appropriate for the new condition needs to be determined. Specific examples include experimental construction prior to executing the main work to check the quality or mix proportion of the solidification materials, mixing and agitation methods, or agitation length and subsequent confirmation of the material, as well as execution methods of the solidification method or its process, or to check the materials and execution methods that may be used when the soil conditions change in the course of work.

In this kind of experimental construction, the major check points basically concern information necessary for construction management such as quality control or work progress control. Even if the check results do not match the original plan, unless a very serious problem arises, it is very rare to change the earthwork structure or soft ground improvement work method. Consequently, solutions generally stop at changes in the materials to use, in the amount of these materials to add, or in the execution procedures (e.g., length of agitation).

3-10-2 Investigation during Work Stage

There are many cases in which the work encounters soil properties that differ from the results of the investigation in the investigation stage because of complicated ground conditions, or when the structure or scale of an earthwork structure is changed during the design or construction stage. In these cases, it is necessary to conduct supplementary soil investigation in the construction stage as required and to reflect the additional data in the improvement work.

Stability control and settlement control are carried out during construction in the event of an earthquake or to measure the behavior of the ground, and make sure that the prescribed safety and settlement characteristics are maintained. The amount of settlement or lateral deformation of the ground is generally measured for stability control or settlement control. Although it is necessary to appropriately determine the measurement items, measurement methods, and measurement locations depending on the status or purpose of the construction work or the surrounding conditions, it is recommendable to perform an advance review of the measurement items and locations by forecasting the behavior of the earthwork structures or the ground in advance with numerical analysis. The details of stability control and settlement control in the construction stage are described in “Chapter 5: Work Control and Maintenance”. When the scale of the work is greater than a certain level, investigations in the construction stage are conducted to realize more rational design and construction of the entire work, as in the case of experimental construction, together with stability control or settlement control at the construction site during the construction work as described above. Specifically, when the work is relatively larger in scale, more accurate and rational design and construction will be realized by subdividing the work site into a number of sections, measuring the behavior of the ground or an earthwork structure under construction in any of those sections where work is conducted ahead of others, and applying the information obtained from this measurement to other sections for feedback in the design and construction. This will make it possible to conduct more rational construction in addition to experimental construction when space is available at the work site or the work period is sufficiently long.