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Manual for Highway Earthworks in Japan

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Synopsis

The highway earthworks technologies in Japan have achieved an extremely high level through long years of research and development and numerous projects undertaken to overcome Japan's harsh natural conditions: frequent earthquakes, heavy rainfall, mountainous topography, fragile geology and soft ground.

However, developing countries, where developing the road network is the top priority to achieve economic growth, are aggressively constructing new roads. On the other hand, due to the natural disasters caused by rainfall, etc., the rapid deterioration of structures and functions of roads forces these countries to spend a large part of their budget to repair damaged roads. A major factor contributing to this problem is the fact that the transfer of technologies for road earthwork has been delayed and almost no progress has been made in establishing standards for earthworks.

In order to contribute to the provision of high quality infrastructure in developing countries by disseminating and transferring Japan's earthworks technologies, we have prepared a manual containing technologies concerning everything from survey, design execution and maintenance in Japan, and have translated it into English.

Keywords : earthwork, survey, design, execution, maintenance, slope protection, embankment, drainage, soft ground

Preface

Civil engineering technologies in Japan have achieved an extremely high level through long years of research and development and numerous projects undertaken to overcome harsh natural, meteorological, topographical, and geological conditions. This is particularly true for road earthwork technologies that are extremely high level technologies reflecting Japan's harsh conditions: frequent earthquakes, heavy rainfall and snowfall, mountainous topography, fragile geology, and soft ground. Since the first publication of the Highway Earthwork Guideline (Japan Road Association) in 1956, we have revised and published separate volumes of this guideline to keep up with technological progress, prepared additional related manuals, and disseminated these road earthwork technologies to engineers throughout Japan through these various technical guidelines and manuals.

However, developing countries, where developing the road network is the top priority to achieve economic growth, are aggressively constructing new roads. But the increase in traffic volumes that accompanies economic growth and natural disasters caused by rainfall, etc. result in the extremely rapid deterioration of the structures and functions of roads, forcing these countries to spend a large part of their budget to repair existing roads. A major factor contributing to this problem is the fact that regardless of progress that these countries have achieved in transferring technologies concerning alignment and other aspects of road structures and establishing standards for such technologies through the process of improving their road networks, the transfer of technologies for road earthwork that includes slope protection, embankment, soft ground countermeasures, retaining walls, drainage and so on has been delayed and almost no progress has been made in establishing standards for such earthworks.

In order to contribute to the provision of high quality infrastructure in developing countries by disseminating and transferring Japan's road earthwork technologies, we have prepared a manual containing technologies concerning everything from the road earthwork survey to its design, execution, and maintenance in Japan and have translated it into English. This English guideline contains the technologies and concepts widely applied in Japan; but does not include the most advanced technologies and concepts that are partially applied. Please refer to related guidelines, documents, and papers concerning these advanced technologies.

In conclusion, we wish to express our heartfelt gratitude to the Committee to Study the Overseas Transfer of Civil Engineering Technology (Earthwork) (Chairman: SHIMAZU Akiomi, Secretariat: Infrastructure Development Institute-Japan) for their generous advice and assistance with the preparation of this English guideline.

The English manual was prepared by translating related passages from the following guidelines and manuals (Japanese language).

- "Manual for Highway Earthworks"; Japan Road Association, 1990
- "Highway Earthwork Series - Manual for Survey"; Japan Road Association, 1986
- "Highway Earthwork Series – Manual for Execution"; Japan Road Association, 1986
- "Highway Earthwork Series - Manual for Countermeasures for Soft Ground"; Japan Road Association, 1986
- "Highway Earthwork Series - Manual for Drainage"; Japan Road Association, 1987
- "Highway Earthwork Series - Manual for Slope Protection"; Japan Road Association, 1999
- "Highway Earthwork Series - Manual for Retaining Wall"; Japan Road Association, 1999
- "Highway Earthwork Series - Manual for Culvert"; Japan Road Association, 1999
- "Design Manual Part 1 (Earthwork)"; Japan Highway Public Corporation, 1998
- "Method of Soil Investigations"; The Japanese Geotechnical Society, 1995
- "Method of Soil Test and Explanation"; The Japanese Geotechnical Society, 2000
- "Design and Execution Manual for Countermeasures for Liquefaction (draft)"; Public Works Research Institute, 1999
- "Manual for River Works in Japan (Design I)"; Japan River Association, 2000

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1. General

1.1 Introduction

1.1.1 Basic Concept of Highway Earthwork

(1) Unique Character of Soil as Construction Material

The subject for earthwork is the natural ground and the main materials to be handled are natural soil and rocks. As the properties of such soil and rocks vary depending on the area, natural conditions and passing of time, engineers involved in earthwork are required to be flexible in the selection of appropriate means to deal with specific site conditions. Compared to the most typical construction materials, such as steel and concrete, there are differences as described below and these differences must be properly recognised for accurate understanding of road earthwork.

(i) Extremely large handling volumes of soil and rocks

It is common for the earthwork cost to account for a large proportion of the total construction cost even though the unit cost of earthwork is relatively low.

(ii) Great variety of soil properties depending on area

Accurate judgement of the properties of local soil to employ the type of work which is appropriate for these properties greatly determines the quality of an earth structure and its construction cost. Accordingly, there is a high degree of reliance on the experience and proper judgement of engineers.

(iii) Dominance of weather conditions

Changes of the water content of soil in particular considerably determine the work conditions. Moreover, the impacts of the water content are strongly felt in regard to the stability and bearing capacity of the embankment made.

(iv) Progressive deterioration of strength with exposed soil and rocks

It is common for newly exposed soil and rocks as a result of cut slope to undergo a progressive deterioration of their strength with the passing of time due to the release of stress and weathering. Accordingly, the stability of cut slope tends to decline at a speed which is determined by complex interactions involving the rock properties, situation of soil sedimentation and weather conditions.

(v) Relatively low strength of soil and erodibility of soil by water

It is unavoidable for small-scale failures to occur with rainwater. However, one characteristic of soil is that partial repair or reinforcement can be conducted at almost any time. Given the advantage of soil's relatively large flexibility, earth structures are regarded as structures of which the functions at the time of original construction can be improved through staged repair and/or reinforcement during the period of their actual use.

(vi) Difficulty of quantitative evaluation of structural stability

It is not easy to quantitatively evaluate the stability of earth structure with soil and rocks based on survey and/or test results. In other words, verification against past performance or experience is strongly required compared to other construction materials because the theoretical elucidation of soil mechanics is still progressing.

(2) Basic Concept

The principal mission of such earth structures as cut slope, embankment, retaining walls and culverts constructed by road earthwork is to perform the function of a reliable foundation for the pavement to ensure the smooth and safe traffic by withstanding the traffic load during the service life. In addition, these structures are required to perform the role of minimising damage to roads, human lives and assets around roads by disasters of various scales caused by natural phenomena, such as rainfall and earthquakes for a long period of time from construction and throughout the service life. Moreover, harmony between earth structures and the surrounding environment is an important evaluation item apart from the necessity to consider the natural and social environments in neighbouring areas during the road construction period. Based on the characteristics of soil and rocks as construction materials, there are many important issues which form the basis for judgement by engineers involved in road earthwork as listed below.

(i) One thing which must not be forgotten in the survey and design of any earth structure is a macroscopic evaluation from the viewpoint of the wide area topography and geology prior to a microscopic survey, testing and stability analysis focusing only on the mechanical properties of the soil. It is common for factors relating to the working of nature to likely dominate the stability of cut slope and natural slopes.

(ii) There has been growing emphasis on empirical technologies for the design of earth structures. Examples are standard slope gradients for embankment and cut slope. These standards for slope gradient to avoid serious disruption to road traffic under the natural environment in Japan are introduced for each type of soil or geological feature based on past performance. Therefore, these standards are deemed to have considered the effect of the rainfall and earthquake apart from extraordinary level. While it is obvious that the

application of empirical technologies within the scope of their applicability would be acceptable, it must not be forgotten that the social and technical demands for roads today are quite different from those at the time of the development of such empirical technologies. To be more precise, there are many instances of embankment or cut slope where the height is beyond the application limits of conventional, i.e. empirical, technologies and the number of cases where uniform judgement of the relative importance of nearby facilities is impossible has been increasing. Although there are many uncertainties regarding the stability analysis of earth structures in extraordinary condition, the maximum efforts must be made to examine slope stability in cases where the conventional application conditions are exceeded.

(iii) There is a conventional idea that the gradual functional improvement of earth structures by means of maintenance and repair work while economising on the initial construction cost is a good policy because of the relative ease of conducting the post-completion structural reinforcement of these structures. However, there is much stronger awareness in recent years than was previously the case of the necessity to ensure a high level of road structures from the beginning to secure the networking function of roads. Labour saving and the cost reduction of road maintenance and repair work will become unavoidable issues in the coming years. Because of this prospect, it is desirable to make roads disaster-resistant with a view to reducing maintenance and repair work at the construction stage of earth structures in the future.

(iv) When embankment is constructed on soft ground, settlement and/or deformation may occur and continue for a long period of time. Even though these changes can be either accelerated or alleviated by the implementation of countermeasures, the expense will be fairly large. Adjustment between the work period (time) and cost is particularly important for earthwork on soft ground.

(v) The consideration of water is the most important aspect of earthwork. It is no exaggeration to say that the workability and stability of embankment as well as cut slope are determined by water. Much effort to lower the groundwater level to reduce the water content of the soil at a work site as much as possible is required in addition to the drainage of rainwater, surface water and spring water from the said site. Moreover, as it is easier to reduce the soil's water content in summer when there are high temperatures, scheduling of the peak earthwork period in summer is effective.

(vi) It must not be forgotten that the accumulation of routine work, such as dealing of water and adequate compaction, and the accumulation of detailed care are the keys to determining the quality of earthwork. For the proper implementation of such work, the mental attitude of always maintaining a tidy work site is essential and this attitude will lead to the prevention of accidents or disasters which often occur at earthwork sites.

(vii) At the connecting sections between rigid structures, such as abutments, culverts and retaining walls, and embankment, careful work is particularly required to reduce the deformation of backfilling parts to achieve good continuity to rigid structures. As the work space at such sections tend to be limited, careful planning of the construction method as well as the materials to be used is desirable. In the case of culverts, the employment of a type of foundation which undergoes deformation in unison with the deformation of the embankment should be actively considered if such type of foundation is structurally permissible.

(viii) During earthwork, an encounter with a situation which has not been predicted at the survey stage often occurs. In such a case, the new ground conditions must be thoroughly surveyed and the survey results must be reflected on the revised design. Information on the soil properties established at the survey and design stages of earthwork should not be treated as being perfect. Instead, this information should be treated as imperfect information which is subject to further improvement by reviewing the conditions at the design stage based on close observation at the construction stage. This idea underlies the method by which the field observations in earthworks on soft ground are fed back to the design in order to ensure work safety and settlement control.

(ix) Even if earthwork is very carefully planned, there is a possibility of the collapse of earth structures under an unexpected situation during the work period. Such accidents tend to occur with temporary structures of which the design safety factor is limited. Design consideration to localise possible damage and measures to deal with damage should, therefore, be examined in advance, assuming cases where collapse and other types of disaster occur due to an extraordinary situation.

1.2 Earthwork Planning

1.2.1 Important Points for Earthwork Planning

(1) Relationship between Road Planning and Earthwork Planning

When planning the construction of a new road, the initial focus is placed on the examination of the essential road functions, i.e. means of transport and transportation. However, the multi-faceted functions and roles of a road mean that there are many diverse related issues to be surveyed and analysed and the required survey and analysis levels have become much higher in recent years.

What is particularly important for the preparation of a good road planning is the integration of all of the required road functions while well balancing those conditions which may be contradictory.

The first stage of road planning is the formulation of a road project plan. At this stage, there are three major issues to be examined: ① road functions, ② social constraints and ③ constraints posed by construction technologies. The comprehensive examination and coordination of these issues are attempted at this stage. An earthwork planning is closely related to all three issues. Particularly in recent years, earthwork has become much larger and speedier while requiring new ideas and technologies in relation to environmental conservation, disaster prevention measures and safe work execution. Multi-faceted examination at this stage is crucial to determine the direction for the planning of earthwork.

The second stage after initial project planning is the determination of the concrete construction method. At this stage, the most rational road plan is prepared by sorting out the preconditions and restrictive conditions and examining the concrete design and work execution method which conform to the design and work execution standards and others. When planning such structures as bridges and tunnels which are not subject to earthwork, it is still necessary for the planning to pay proper attention to earthwork of approach road. As the earthwork cost accounts for a large proportion of the total road construction cost, careful examination of the earthwork planning is essential to ensure efficient project execution. It must be carefully noted that the results of earthwork have a major impact on the quality of a whole road plan.

When a new road is to be constructed at sections with large-scale slope failure, rockfall and debris flow, etc. where ordinary control measures may be insufficient, it is desirable to change the route or to opt for such structures as a tunnel or bridge at the planning stage.

(2) Earthwork at Each Stage of Road Construction

The processes to construct and operate a road can be divided into the survey/design, construction and maintenance stages. An earthwork planning must be prepared to ensure the rational and smooth progress of the work as intended at each of these stages.

① Survey/Design Stage

At this stage, the collection of existing data and site reconnaissance, etc. is conducted to obtain information and data required for project execution. Using such information and data, a draft plan and an alternative plan(s) are prepared for comparison from the technical, social and economic viewpoints to analyse the project feasibility. The draft plan gradually becomes more elaborate and surveys are added in line with the

maturing of the plan. At this stage, negotiations with related public bodies and local communities are concurrently taking place to prepare a rough plan. At the design stage, the rough plan is elaborated to the detailed design and quantity calculation and cost estimation are conducted, leading to the placement (receipt) of a work order. At this stage, the acquisition of right-of-way and various compensations is an important element and the establishment of clear prospects for these issues is essential to ensure smooth work progress. The rough plan prepared at the previous stage is modified to reflect the real conditions and appropriate construction methods and work processes are determined for preparation of the design documents.

② Construction Stage

At the construction stage, it is essential for the designed work to be executed as planned by the contractor so that the target objects are constructed in the right places with the pre-determined function(s) and quality. The contractor in charge of construction work must prepare an optimal construction plan to commence the work. In the case of earthwork, as it is not easy to fully understand the soil conditions with a preliminary survey alone, it is not unusual for unexpected circumstances, including weather conditions, to arise. In this case, the introduction of expedient measures, including additional surveys and necessary change of design, is particularly important.

Execution management is an important requirement for the contractor to construct the object(s) demanded by the owner and the contractor conducts the related various tests and measurements. Although execution management is executed by the contractor to ensure the complete construction of the object(s) demanded by the owner, the contractor often consults with the owner on the work execution method and other issues for the latter's approval. Meanwhile, the owner conducts supervision and inspection to ensure the construction of the object(s).

There are important points to note in relation to execution for construction work, including earthwork. One is the so-called on-site production of a single product, i.e. the construction of a specified object(s) at a specified site(s), unlike the production of items at ordinary factories. Another is the difficulty of execution management as construction work is liable to impacts on the natural and social environments, a fate of construction work. Good execution of the work is only possible when supervision by the owner and execution management by the contractor are mutually complementary through the cooperation of the two parties.

③ Maintenance Stage

Finally, after the completion of construction work, the owner conducts an inspection to confirm that the specified object has been constructed and maintains the object after its handing over by the contractor. Maintenance must ensure that the object fully performs its functions safely for a long period of time and preserves its stability as a structure. For this purpose, regular inspection and monitoring are essential to check the behaviour and deformation of the structure, if required, to introduce corrective measures.

(3) Consistency from Planning to Management

Transformation of the land due to development activities and the changing sense of values of the general public have been steadily increasing and diversifying the social demands for roads. The effective development of road network under these circumstances requires a consistent view of planning through management in all road-related fields, including earthwork.

For example, the planning contents for a road to be constructed by cutting open a mountain area in the case where the earthwork plan is prepared based on the assumed development of the surrounding area in the future clearly differ from the case where such development is not taken into consideration. In addition to consideration of the landscape, such differences include the layout of drainage facilities. Regarding construction work, the absence of the construction of appropriate drainage facilities has adverse impacts on not only the work schedule but also on the stability of slopes outside the work area. Consistency is also required for post-completion maintenance. There have been cases where the lack of appropriate planning and design of earthwork has made maintenance very difficult even if it has not led to a disaster.

Disasters caused by downpours in mountain areas are often related to the original planning. While it is not easy to introduce sufficient preventive measures because of constraints on the initial investment for road construction, there is a growing need for earthwork plans which carefully take improved stability and environmental conservation into consideration.

It is pointed out that the stability of earth structures is generally inferior to that of bridges and tunnels. Obviously, the presence of weak points in parts of a road as a continuous structure is undesirable and well-balanced stability throughout a route is crucial. For this reason, extra care is required to ensure the safety of earth structures. Given the need for labour saving in future road maintenance work, the construction of earth structures which are safe enough not to require much maintenance work except at times of a downpour or earthquake involving extraordinary force should be pursued.

(4) Consideration of Natural Environment and Landscape

In areas which are important for conservation of the natural environment, it is important to minimise alteration of the environment by a road construction. For this purpose, proper consideration should be given to the selection of a route which avoids areas in need of preservation or conservation and to the avoidance/minimisation of possible alterations through the skilful use of tunnels and bridges. In regard to slope protection work, the adoption of appropriate techniques depending on the surrounding conditions of the site is necessary while ensuring the stability of slopes. These techniques include curtailment of the earth volume and slope area by means of careful examination of the horizontal and vertical alignments, rounding of slopes, modification of the slope gradient and the adoption of revegetation work to harmonise with the environment.

Road planning in Japan which has many mountainous areas often involve the creation of slopes. It must be noted that consideration of the landscape involves not only the appearance of slopes but also the appearance of roads and the view from roads. The harmonisation of road structures with the surrounding environment through revegetation work or the hiding of such structures by revegetation work are effective measures together with the careful design of the shape and surface of structures. However, the adoption of such techniques as the modelling and painting of structures should be carefully analysed as the evaluation results of their value tend to vary.

1.2.2 Proceedings of Earthwork

The proceedings of road construction have been discussed so far in terms of the stages involved. In this section, the actual proceedings are analysed in further detail and their connection with earthwork are described. Needless to say, there is no set formula for the proceedings of road construction and the actual proceedings vary depending on owner, type of project, scale of project and area. Fig. 1-2-1 shows the typical flow of national road construction work in Japan. The activities to be conducted at each stage are described below in accordance with this flow and with special emphasis on earthwork.

(1) Planning Survey

A planning survey is conducted to draw many alternative routes while meeting the road structure criteria as determined by the characteristics of and relevant standards for the proposed road and to finalise a single route. Multi-faceted examination is conducted in view of the social, topographical and geological conditions of areas along the proposed route. It is, therefore, necessary to survey the situation of land use, presence of cultural assets, related public works in progress and environmental issues together with a full hearing of the

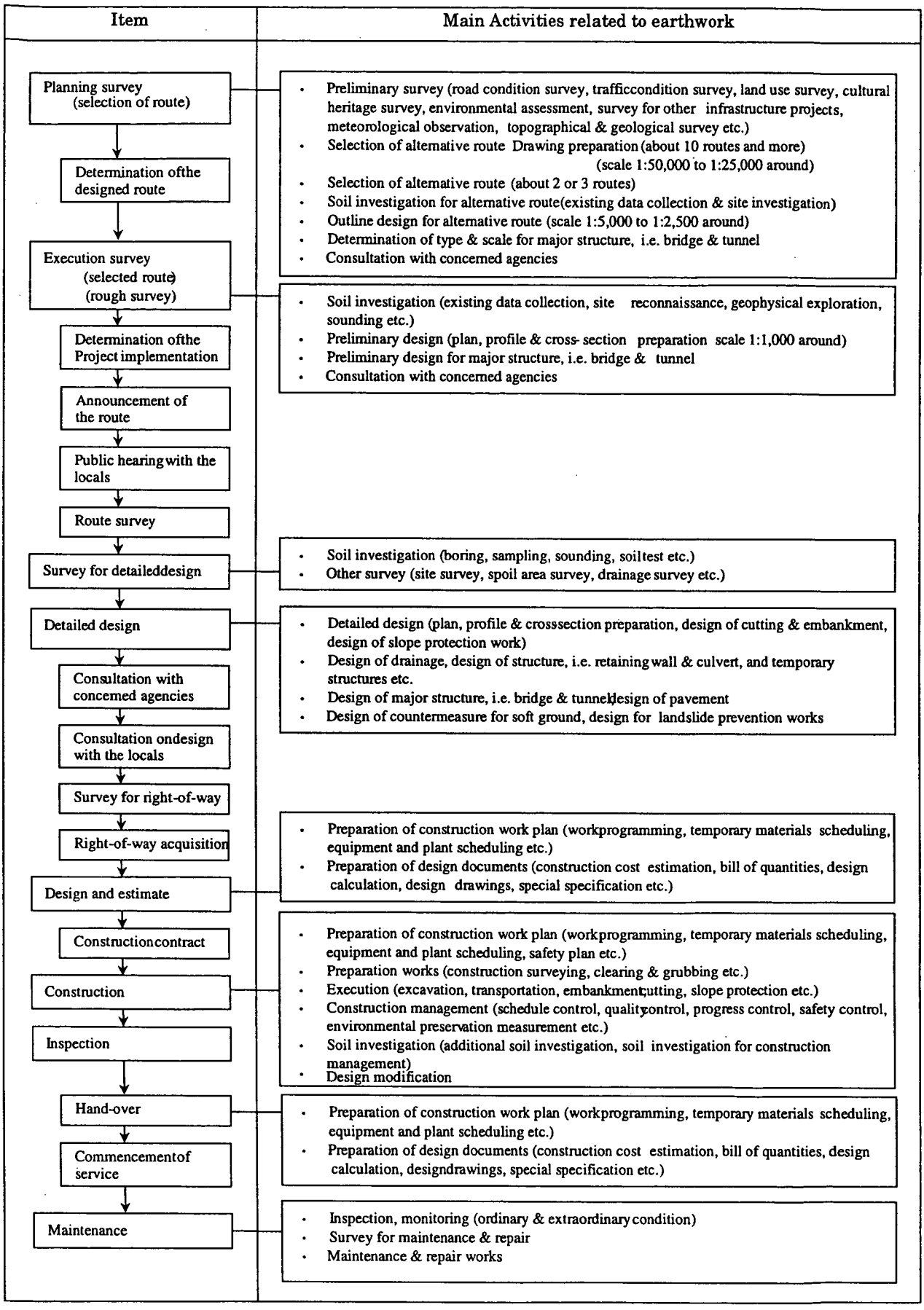


Fig. 1-2-1 Flow Diagram of Road Construction

opinions held by local public bodies as well as local residents and consultations with bodies related to the planned road construction.

Information of the local soil is required to produce a rough structural design of the road. The use of existing materials and site reconnaissance are the main activities at this stage and a full-scale soil survey is not usually conducted.

The earthwork planning at this stage outlines the basic cross-section, earthwork section(s) and appropriate cutting and banking volumes and the basic profile for road construction work along the route is established. Accordingly, any problem relating to the topography or geology is identified. What is important here is avoiding places where construction work is expected to be difficult. The forced inclusion of these places in the plan carried difficult problems forward to the post-completion maintenance stage. Particular attention should, therefore, be paid to mountainous areas with a steep gradient, landslide areas and talus topography which form the control points for route selection from the viewpoint of earthwork planning.

(2) Execution Survey

An execution survey is intended to give concrete form to the plan and to transform the survey findings into actual project components. As such, this is the most important work at the survey stage. However, it is often the case that the topographical and geological data required for road design is difficult to obtain by means of surveying and a geological survey on site because of the circumstances surrounding the local communities which will be affected by a proposed road and other reasons. In order to produce a preliminary design of which the accuracy is higher than that prepared at the above-mentioned planning survey stage, existing materials and maps based on aerial photographs, etc. are used for this survey together with efforts to obtain as much detailed information as possible, including site reconnaissance results and sounding results, if possible. At this stage, it is still not possible to formulate a detailed earthwork plan and the main activity is examination of the work difficulty and level of stability during the work and after completion from the viewpoints of engineering and environmental conservation. One essential task for this examination is to at least ensure that the problems identified by the examination will not become fatal shortcomings which hamper the progress of road construction. Even if measures to rectify these problems are available, they must be economically viable. Special attention should be paid to the need to examine (i) the suitable cross-section for major cut slope and embankment, (ii) the drainage system, including waste water disposal, at the end, (iii) the stability of large-scale slopes and the suitability of protection works for such slopes and (iv) the approach to such structures as bridges and tunnels as problems tend to occur with these aspects of road construction

work. The execution survey is then followed by announcement of the proposed route based on the findings of this survey and hearing of local opinions.

(3) Survey for Detailed Design

Announcement of the proposed route is usually followed by consultations with local communities and the commencement of surveying and soil investigations along the route. At this stage, center pegs of alignment are placed, longitudinal levelling and cross-sectioned survey are conducted and soil investigations involving on-site boring, soil sampling and in-situ test is conducted. The detailed design is then conducted based on the findings of these activities and concrete data on the construction sites, borrow pits and spoil areas. As the detailed design is required to have sufficient accuracy as the basis for the placing of width pegs, land acquisition or order placement for construction work (cost estimation), it is essential to obtain all of the necessary data and reference materials in advance. Given the fact that an earthwork planning is often severely restricted because of failure to secure a spoil area, careful examination in this regard is essential at this stage.

(4) Detailed Design

At the detailed design stage, countermeasures for soft ground, cross sectional configuration of cut slope and embankment, such structures as retaining walls, culverts, viaducts and bridges and slope protection, etc. are designed based on the soil investigation results. In regard to earthwork, an appropriate design involves a balance between cutting and embankment volume and examination of the stability and workability of cut slope and embankment in view of the site topography and geology. As earthwork is strongly affected by water, including surface water and groundwater, a drainage plan for during and after the work should be carefully prepared. One especially important point here is to give transverse channels a sufficiently large cross-sectional area to avoid insufficiency in the future due to an increase of the discharge with the development of the drainage basin. It is also necessary to examine suitable construction roads from borrow pits and to spoil areas, temporary roads during the work and a temporary work plan for earth retaining and cofferdams, etc.

Another important point for the design work at this stage is to make it easy to inspect and repair all of the structures to ensure easy road maintenance.

(5) Cost Estimation

At the cost estimation stage, the route is divided into work sections and cost estimation is conducted for each work section based on the detailed design for the placement of an order for construction work. Firstly, the schedule for each type of work is determined based on the soil investigation results and other relevant information while outlining the contents of the construction plan, including scheme of haul, selection of the construction machinery to be used, calculation of the work capacity and preparatory work. The cost for each type of work is then estimated and totalised.

(6) Construction Work

As earthwork involves many factors, such as the geological conditions which are uncertain and which are difficult to establish in advance, it is important to introduce a regime under which the site conditions are constantly monitored with a view to conducting a survey on or analysis of any problem which is encountered so that an appropriate judgement can be made and remedial measures can be quickly implemented before a problem is beyond remedy.

Prior to the commencement of construction work, the design documents, drawings and specifications should be carefully examined and the site conditions should be accurately identified. In addition, the relevant data and information produced by the surveys implemented so far and the local situation, etc. should be carefully analysed to build up a precise understanding of the concepts harboured by the designers.

For cutting or banking work, it is essential to fully investigate the relationship between groundwater/surface water and the soil properties. As the workability and work quality can be significantly affected by the work timing due to a substantial rise of the groundwater level and rainfall, the timing of order placement for construction work and of the construction period must be properly arranged. In the case of road construction work at mountainous and hilly area, the actual geological features of cut slopes and the foundation ground of structures such as retaining walls may not necessarily be as previously anticipated. The encountering of problematic geological features, including dip slopes, is not a rare occurrence. A flexible stance to deal with these problems as they arise is required and the implementation of a soil investigation, including boring and test pitting, with a view to examining possible countermeasures is particularly important to ensure the safe progress of the work.

(7) Inspection

Inspection is conducted to check whether or not the entirely or partially completed work meets the contract conditions for the purpose of deciding the pass or failure of the work in question. Inspection may also be

conducted at a certain stage of the work to determine whether or not it is justified to advance to the next stage or in the case where verification of the work quality after the completion of the work is difficult. Judgement on pass or failure is conducted for both the finished work quality and the quality of the completed structures. Here, the finished work quality means the location, dimensions and quantities of the earth structure(s) created by the work while quality of the completed structures means the degree of compaction and strength of the soil as a material.

Inspection usually takes the form of sampling, supplemented by visual observation and other means. The inspection method to be adopted must be simple, effective and highly accurate, taking the work contents, required time and cost into consideration. For example, the number of inspections for the finished work quality must be increased in places where the cross-section significantly changes. When visual observation finds places where the continuity of the work is poor, appropriate measures, including additional measuring, should be implemented. In regard to the test on the degree of compaction as part of the quality inspection, special attention should be paid to those places which are likely weak points of the earthwork, such as near the slopes of an embankment or near structures, to ensure effective inspection.

(8) Maintenance

Once completed, a road with its structures is handed over to the owner. However, adequate maintenance is required for this road to subsequently perform its specified functions for a long period of time.

The monitoring regime and method to constantly maintain the road in good condition may be described in the patrol guidelines prepared by the road administrator. In the case of national roads for example, these are subject to several types of patrols: routine patrols (at least once a day), night patrols (at least once a month), periodic patrols (at least once a year) and emergency patrols. For earth structures, routine patrols and periodic patrols are required to inspect the retaining walls, culverts, slopes and drainage facilities.

A downpour or earthquake may cause slope failure, soil outflow due to clogging of the drain ditches, the collapse of an embankment or retaining wall due to poor drainage, scouring of the wing walls at the intake or outfall sections of the transverse drainage channel and/or settlement at the earthwork sections of a road.

Moreover, scouring of a road often takes place due to the massive overflow of rainwater from drainage ditches or the inflow of a large volume of water from an access road, etc. in a very short period of time, in turn caused by the insufficient capacity of the drainage system. Soft ground sections and connecting sections between cut slopes and embankments tend to suffer from faulting or differential settlement with the passing

of time even if the work is carefully executed, creating maintenance problems for a long time to come. At soft ground sections where residual settlement often occurs, it may be necessary to change the pavement composition, including the introduction of provisional paving, and to readjust the timing of paving work. While some of the problems mentioned above cannot be predicted in advance, others may be preventable with the implementation of appropriate measures during construction work. What is required is consideration of the long-term maintenance requirements at the survey, planning and design stages. Work records, design drawings for construction work and soil survey data, etc. must be routinely filed and kept as they are useful for the maintenance and repair of earth structures and also for the planning of post-disaster rehabilitation measures.

1.3 Explanation of Technical Terms

The names of the various components of earthwork and the definitions of the main technical terms used in the present Guidelines are given below (see Fig. 1-3-1 and Fig. 1-3-2).

(1) Banking Section

A road section created by banking on the original ground because of the higher subgrade level than the original ground level is called a banking section.

(2) Cutting Section

A road section created by cutting the original ground because of the higher original ground level than the subgrade level is called a cutting section.

(3) Embankment

The part of a banking section where the soil from the existing ground level to the subgrade level is banked is called an embankment.

(4) Cut Slope

The part of a cutting section where the existing ground is excavated down to the subgrade level is called a cut slope. Even if the subject ground consists of rock instead of soil, the uniform description of cut slope is used.

(5) Filled Up Ground

The soil part of an embankment other than the subgrade is called filled up ground.

(6) Subgrade

The subgrade is an almost uniform earth layer of a depth of approximately 1 m below the pavement and provides the basis for the decision on the paving thickness. This layer corresponds to the upper part in the case of a banking section or the some 1 m deep soil layer below the original ground excavated to a specified level in the case of a cutting section. The localised part where the subgrade soil is replaced to obtain a subgrade layer with a uniform bearing capacity and the backfilled transition part of the connecting section between a cut slope and an embankment are included in subgrade.

(7) Pavement

The pavement is the part from the concrete pavement slab surface to the base course in the case of a cement concrete-paved road or the part from the surface course to the base course in the case of an asphalt-paved road.

(8) Slope

A slope may mean an artificially created slope by road earthwork, i.e. a cut slope or an embankment slope, and may also include, in a broader sense, a natural slope of the natural ground. In normal circumstances, a slope here means a cut slope as well as an embankment slope.

Meanwhile, slope protection work means the covering of a slope by vegetation or a structure for the prevention of erosion, weathering or the slope failure. Rockfall prevention work, however, is not included in slope protection work.

(9) Retaining Wall

A retaining wall is a type of structure to support the soil to prevent collapse of the soil and is constructed when soil alone cannot provide the required stability for earthwork because of conditions of right of way or topography. There are several different types of retaining walls, including concrete block retaining walls, gravity retaining walls, cantilever retaining walls and reinforcement soil walls.

(10) Culvert

A culvert is a type of structure which is created through embankment or the original ground to secure the space for a road or water channel which crosses under a road. Based on the mechanical characteristics, culverts are classified as either rigid culverts or flexible culverts. There are many types of rigid culverts, including box culverts, portal culverts, arch culverts and pipe culverts, while the types of flexible culverts include corrugated metal culverts and rigid polyvinyl chloride pipe culverts.

(11) Temporary Structures

Temporary structures are structures which are temporarily built to construct the permanent structures of a road and include earth retaining, cofferdams and temporary piers.

(12) Drainage

Drainage from a road takes many forms, including surface drainage, sub-surface drainage, slope drainage, drainage from the backfill of a structure or from inside a structure and transverse drainage of a road such as a culvert depending on the target water for drainage.

(13) Soft Ground

Soft ground is a type of ground which consists of a soil formation made up of such soft soil as clay and silt with a high proportion of fine particles, organic soil or peat with large voids or loose sand and is prone to problems of stability associated with a generally high groundwater level as settlement is likely to occur due to the load of an embankment or structure.

(14) Rock fall

Rock fall is a phenomenon where foliated rocks and gravel due to enlarged cracks (caused by joints, foliation and stratification developed in the bedrock) in the bedrock or outcropped rocks, boulders and gravel originally contained in talus material, pyroclastic material or a gravel bed with low degree of compaction start to fall down a slope.

(15) Landslide

A landslide is a phenomenon where the soil mass above a boundary deep in the ground gradually shifts downward. Landslide sites tend to be concentrated in areas with specific geology or geological structure. Compared to failure of cut slope or a natural slope, the gentler slope moves on a large-scale, forming specific topography (landslide topography).

(16) Debris Flow

Debris flow is a phenomenon where soil and boulders are liquefied by surface water or groundwater and tend to flow downward through a mountain torrent. It usually has huge energy and destructive force. Debris flow tends to occur in places where there is a massive sediment of unstable debris along a steep torrent or when there is a large risk of slope failure due to heavy rain in the catchment basin.

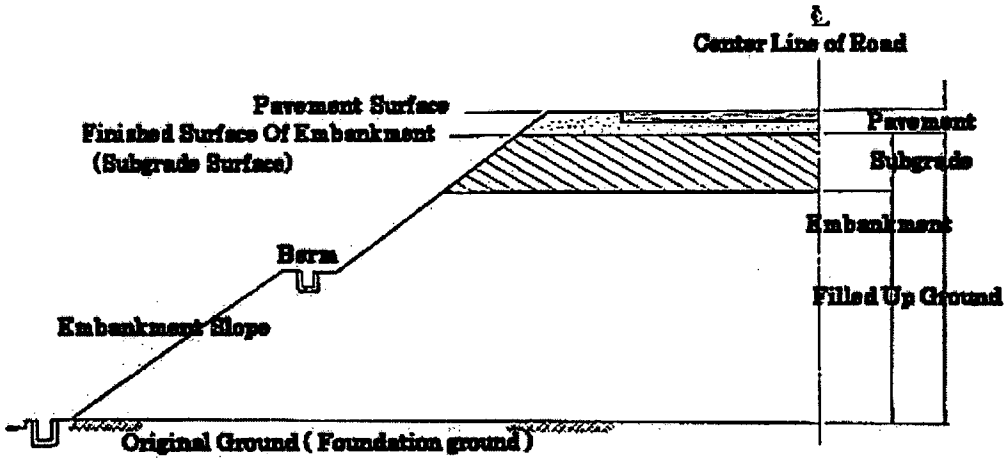


Fig. 1-3-1 Cross – Section of Embankment

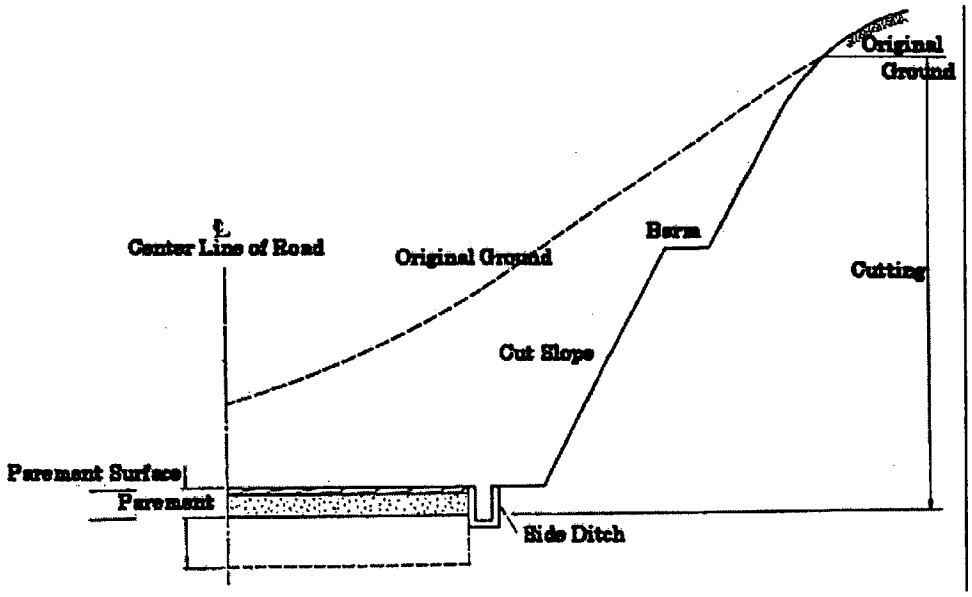


Fig. 1-3-2 Cross-Section of Cut Slope

2. Surveys

2.1 Introduction

2.1.1 Relationship between Procedure of Road Construction and Soil Investigation

Fig. 1-2-1 indicates the flow of road construction and the survey required at the various stages of such construction. As can be seen from this figure, the types and the required accuracy of the survey differ by the stage of the construction and it is necessary to undertake appropriate surveys using appropriate methods with consideration given to the local conditions and characteristics of the road in question.

Surveys that are considered closely linked to road earthwork may be categorized into soil investigation (including topographic survey and geological survey) and other surveys (survey of the weather, environment, etc.) and the soil investigation is the most important from the engineering view. For this reason, the basic policies in undertaking such surveys will be detailed including points and matters to be noted in such factors as cut slope, embankment, drainage, soft ground, rockfall, landslides, debris flow, retaining wall and culvert. Only simple explanation is provided on survey other than soil investigation.

Detail of the method of investigation and test are referred to related standards.

2.1.2 Survey Method

(1) Preliminary Survey

Collection of relevant existing data (topographic maps, geologic maps, geological and soil investigation reports of other construction works in the vicinity, construction records, disaster records, etc.), interpretation of aerial photographs and site reconnaissance are the principal activities undertaken as preliminary survey.

However, for areas that may exert significant impact on the route planning, the road structure and construction cost due to such factors, failure and landslide prone areas, soft ground, areas where large scale ground cutting is expected, bridge or tunnel construction sites and areas where significant drying up of ground water through ground cutting is expected, geophysical exploration, sounding or boring ought to be undertaken to the extent possible.

(2) Main Survey

The main survey is undertaken without fail prior to detailed design and in addition to performing an overall survey of the soil and geological conditions of the entire route, the purpose of the survey is to clearly identity the soil and geological conditions of areas that may present problems in the road construction. The main survey is undertaken for the entire route of the road.

In selecting concrete points at which the survey is implemented, it is desirable to prioritize sections that need particular review from the perspective of the road structure and topography. However, since it is difficult to obtain all information concerning soil and geological properties through a single survey and since the road structure is not clearly determined at the time the survey is undertaken, the soil and geological conditions should be identified in a general manner by first undertaking sounding, boring, sampling and testing of soil and rock properties at appropriate intervals along the route. Moreover, in the event the road structure is changed as detailed design of the road progresses, further prioritized survey should be undertaken with respect to places where the soil and geological conditions or the topographic conditions are complex, places where large or special structures are expected.

The main survey generally involves boring as the primary method. In principle, the depth of the boring survey is up to a depth where the bearing stratum is identified to a thickness of 5 meters in normal locations. However, in locations where ground cutting is to be implemented, boring is performed to a depth of 2 meters below the road surface and in locations prone to landslides, boring is performed to the deeper of the assumed sliding surface or 5 meters below the determined subgrade surface. Moreover, for the foundations of structures other than brides, the N value shall be 15 or more and for the foundation of bridges, boring shall be performed until adequate thickness in the bearing stratum required for the assumed structure is confirmed. Clayey ground with N value of 4 or less should be considered soft ground and investigated. Sandy ground with N value between 10 and 15 or less should also be investigated with respect to possible liquefaction.

2.2 Survey on Cut Slope

2.2.1 Checkpoints of Survey

Generally, the geological structure and properties of soils change considerably for ground which has been cut to form the slopes and so it is extremely difficult to determine the detailed properties and characteristics prior to the execution of work and it is especially difficult to theoretically review the stability of cut slopes from the results of the soil and geological surveys. Nevertheless, it is required to determine the properties of soil and rock, the

presence or absence of landslide and slope failure hysteresis and the influence of groundwater by performing soil investigation and geological surveys at some typical points in the sections to be worked out.

Especially, the design of large cut slope in particular, greatly affects not only the construction cost but may also frequently cause slope troubles during construction and maintenance. Therefore, appropriate surveys suited to the purpose must be selected and performed. It is required to survey and test the excavated soil if it is to be used as banking materials.

The main checkpoints of surveys for cut slope are:

① Stability of cut slope
② Stability of adjacent natural slope (see 2.6 to 2.8 for rock fall, landslide and debris flow)
③ Difficulty in excavation
④ Change of groundwater level in adjacent ground due to excavation (see 2.4)
⑤ Soil property as the banking materials (see 2.3)

Appropriate surveys should be planned for putting stress on items which have some problem and taking account of the economic aspect in such a manner that the purposes of the survey can be satisfied.

2.2.2 Cut Slope and Natural Slope Failures

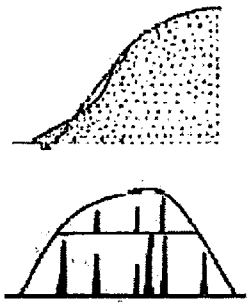
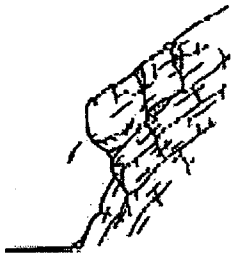
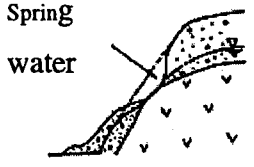
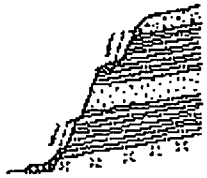
Cut slope and natural slope failures have different shapes based on the principal conditions, i.e. soil, geological structure and groundwater conditions and forth, and the secondary conditions, i.e. magnitude of cutting, shape of slope and rainfall and forth.

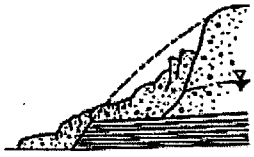
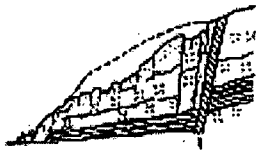
Slope failures can be roughly classified into the following types as shown in Table 2-2-1.

2.2.3 Survey on Cut Slope Stability

In the case of cut slopes, the ordinary survey should begin with the preliminary survey described in 2-1-2. This preliminary survey is performed to study the actual condition of the existing slopes, to judge the presence or absence of landslide, debris flow and rock fall hysteresis and also to determine the groundwater conditions based upon the rough survey on geology, soils and their structures. The results of the survey are used for determining the rational survey plan for the main survey to be performed in the next stage.

Table 2-2-1 Classification of Cut Slope and Slope Failures

Category	Description	Schematic Drawing	Typical Geological Features	Remarks
(1) Erosion/ Collapse	1) Exfoliation of the surface or formation of a gully due to dry-wet repetition, freezing and rainwater erosion, etc; may develop into a deep failure if not arrested	 Plan	Volcanic ash soil; decomposed granite soil; tuff (Neogene); weathered slate; top soil	Often occurs at simply cut slopes or planting with a poor survival rate
	2) Collapse of an overhang section at a slope		Terrace gravel; colluvial soil; pyroclastic material	Likely to occur at natural slopes but also occurs when the lower part of a cut slope slides down
	3) Collapse of rocks with many cracks and/or joints		Mesozoic and Paleozoic formations; igneous rock	As above; a small-scale collapse involving the falling of individual rocks is classified as rockfall while a large-scale collapse is classified as a bedrock failure
(2) Surface Failure	1) Slipping of the top soil; sometimes includes a heavily weathered rock layer; often induced by spring water		Sandy soil clayey soil, colluvial soil, heavily weatherd layer of granite, tuff, mudstone, slate and andesite, etc.	
	2) Slipping of the surface rock layer due to weathering, etc.		Mudstone, tuff and diabase and weathered slate and schist, etc.	Special attention is required for cut slopes as rapid weathering takes place

	3) Slipping of rocks along a dip slope structure or cracks of the rock base (joints, small faults or thin layer); often a wedge-shaped failure in the case of the latter	 	Rocks with a dip slope structure (alteration of strata, crystalline schist and slate, etc.) and rocks with well-developed joints, bedding stratification or schistosity (slate, crystalline schist, serpentinite, granite, rhyolite, andesite, chert and quartz porphyry, etc.)	
(3) Large-Scale Failure/ Landslide	1) Large-scale slipping of a slope formed by soft and not well consolidated soil or a slope with an unstable factor in terms of the geological structure due to a rise of the groundwater level	  Lower layers with different permeability	Thick deposit of a talus layer, colluvial soil, gravel and volcanic ash soil, etc. on the sand layer, rock base or old sliding surface	This type of failure often occurs when the quantity of groundwater is particularly large; may lead to a large-scale failure depending on the soil and topographical conditions
	2) Large-scale slipping of a bedrock which has such a geological structure as a dip slope, fault or fracture zone; one type of bedrock failure	  Fracture	Rocks with a dip slope structure (alteration of strata, crystalline schist and slate, etc.); can occur with such igneous rocks as granite and quartz porphyry	
	3) Forward collapse of a support plate-type slope or a slope with well developed cracks; one type of rock base failure		Mesozoic and Paleozoic formations; Tertiary system; igneous rock; serpentinite	A small-scale failure is classified as a surface failure

The main survey is performed for detailed design of the slope and may be made in two or more steps if required. The main survey is to be performed in addition to the results of the preliminary survey to find the detailed condition of strata and the various parameters and information required for designing and reviewing the places that may create stability problems.

The survey on the natural slope above the cut slope is also performed in accordance with the survey procedure for cut slope.

Table 2-2-2 indicates the relationship between the main points of the survey and the survey method at the cut slopes.

Table 2-2-2 Focal Points of Survey at Cut Slopes and Survey Methods

Main Focal Points of Survey \ Survey Method		Site Reconnaissance	Geophysical Exploration: Elastic Wave Exploration	Geophysical Exploration: Electric Survey	Geophysical Exploration: Underground Radar	Geophysical Exploration: Electric Logging	Geophysical Exploration: Velocity Logging / Acoustic Velocity Logging	Boring Survey	Test Pit	Sounding	Borehole TV	Soil Test	Rock Test	Groundwater Survey	On-Site Measurement
Soil Properties and Lithology 1)		◎	○	△			△	◎	△	△	△	◎	◎		
Geological Structure	Geological Structure: Composition of Strata (Alteration of Strata, etc.) / Fault / Fracture Zone / Dip Slope / Support Plate	◎	○	△		△	△	○	△		△				
Geological Structure	Distribution and Nature of Fissures and Cracks	◎	△		△		△	◎	△		○		○		
Geological Structure	Degree of Weathering / Decomposition	○	○	△		△	△	◎	△	○	△		△		
Geological Structure	Thickness of Top Soil / Talus / Colluvial Soil	○	○	△	△		△	◎	△	○	△				
Ground Strength			△				△	○	△	○		◎	◎		△
Deformation, such as Dip and Cracks		◎													○
Situation of Groundwater and Spring Water		○		○				○	△					◎	
Fluctuation of Groundwater Level								○	△					◎	

Note

1) Soil properties means the name of the soil, state of stratification, changing strength in the depth (N value graph), degree of hardness (or softness) and state of compaction while lithology (rock properties) means the name of the rock, state of stratification, degree of vulnerability to weathering and degree of cracking, etc.

2) ◎ most frequently used method; ○ frequently used method; △ supplementary method

Careful thought is required to determine the boring sites and boring should be conducted at problematic sites from the viewpoint of slope stability as found by the site reconnaissance. In the main survey, boring should be conducted along the centre line of the road at sites where the topographical or geological features change. It is desirable to conduct at least two additional borings across the road at sites, which are found to be problematic in terms of slope stability, or at sites with a large-scale slope according to the results of the preliminary and main survey. At sites where elastic wave exploration is conducted, it is desirable for boring to take place at the crossing points of the lines.

2.2.4 Survey on Cut Slope Requiring Extra Precautions

(1) Cuts in Colluvial Deposits and Heavily-Weathered Slope

In the case of cutting unstable ground such as talus ~~cone~~ which has a low degree of solidification, weathered slope, volcanic mudflow, and previous slope failure zones, the site reconnaissance should be performed in the first place. It is required to determine the groundwater level and the N-values from the results of boring, the grain size distribution (condition of matrix) from the results of soil test, and the state of stratification as determined from the elastic wave exploration. In addition, it is required to fully survey the main factors which may govern the stability of slope such as the relation between the depth of cut and groundwater level, the degree of solidification of matrix and grading, dip of bedrock, and the presence or absence of hysteresis of slope failures in the past. The results of this survey should be reflected in the design.

(2) Cuts in Easily Erosible Soils such as Sandy Soil

Sandy soils are easily eroded by the surface water, and sometimes resulting in rock falls, slope failures or outflow of soil. In order to determine the slope gradients for such soils or to design slope protection works, it is necessary to make a comprehensive review of the degree of solidification and the erosion resistance by considering the N-value, and the sand and silt content by grain size analysis and the results of the scour resistance testing and the survey of the actual condition on the existing slopes.

(3) Cuts in Quickly Weathered Rock such as Mudstone and Serpentinite

For slopes cut in rocks with a low degree of endurance against weathering, the appropriate gradient of slope cut should be determined after comprehensively evaluating the velocity of elastic waves through the ground, the unconfined compressive strength of sampled cores, and the relation between the thickness of weathered

zone of existing slopes and the period of time which has elapsed after the cut was made as well as the results of the dry-wet repeating test.

(4) Cuts in Rocks with Numerous Fissures

The stability of slopes cut into rock is mainly governed by the fault or the degree of fracture as well as by conditions of fissures having certain regularity, such as bedding stratification, foliation and joints. The properties of rocks and the conditions of fissures should be fully studied through the detailed observation of the existing slopes and outcrops, and a comprehensive engineering evaluation should be made based upon the core recovery (R.Q.D.), and calculations of index of degree of fissuring in addition to the elastic wave exploration. If the direction of the fissure inclination has a regularity in a certain direction and coincides with the direction of the slope inclination (that is, if there is dip slope), then an extremely unstable slope may have formed, necessitating careful performance of the survey.

(5) Cuts in Ground with a Large Amount of Groundwater

It can be said that most causes of slope failures are related directly or indirectly to groundwater. The groundwater can be determined by various methods such as borings, electric prospecting and pumping tests and thus an appropriate type of survey should be selected and performed taking account of field conditions.

(6) Cuts with Large Slope

Large slope is rarely composed of uniform and rigid soil throughout the slope and more often is composed of weak layers such as weathered layers, faults and altered layers. In particular, when cutting through col, since cols are often fault fractured zones, the degree and direction of the fracture should be confirmed through boring or elastic wave exploration.

Moreover, rocks containing a large amount of montmorillonite that is said to be expansive rock may often be judged to be relatively hard rock through in situ tests (standard penetration test, elastic wave exploration) at points where the depth of the soil covering the rock is large. However, in large slope, a large amount of stress is relieved and significant secondary decline of strength occurs after the ground cutting.

In particular, for large slope, since changes of configuration of slope (re-cutting) during the cutting is disadvantageous form the perspective of construction cost and efficiency of work, detailed survey of the soil property and condition of ground water is required.

(7) Cut Slope prone to Damage by Earthquake

Looking at the damage of cut slopes and natural slopes caused by the earthquake, incidences of surface failure and rockfall are extremely numerous. In particular, in the natural rock slope with overhanging portions, cut slope or natural slope composed of rocks with numerous fissures and slope with boulder stones which is not firmly embedded in the ground, rock fall are prone to occur. The incidence of surface failure of slope is high when the longitudinal or cross section of slope forms protrusions (Fig. 2-2-1). Moreover, there is a tendency for the head of the failure zone to reach the ridge.

Furthermore, deep failure of cut slope and surface failure of natural slope also occur causing serious damage.

Such factors as the height, the slope gradient, presence or absence of overhang, geological conditions of the slope, thickness of the topsoil, presence or absence of spring water, frequency of rock fall and failure, shape of the slope, presence or absence of any unsoundness of deformation in the slope and type of existing preventive work impact the stability of the cut and natural slope and the extent of damage in the event of an earthquake. For this reason, in order to investigate the stability of a cut and natural slope in the event of an earthquake, careful examination of aerial photographs and site reconnaissance need to be undertaken and boring and sounding to be performed as required in order to review countermeasures.

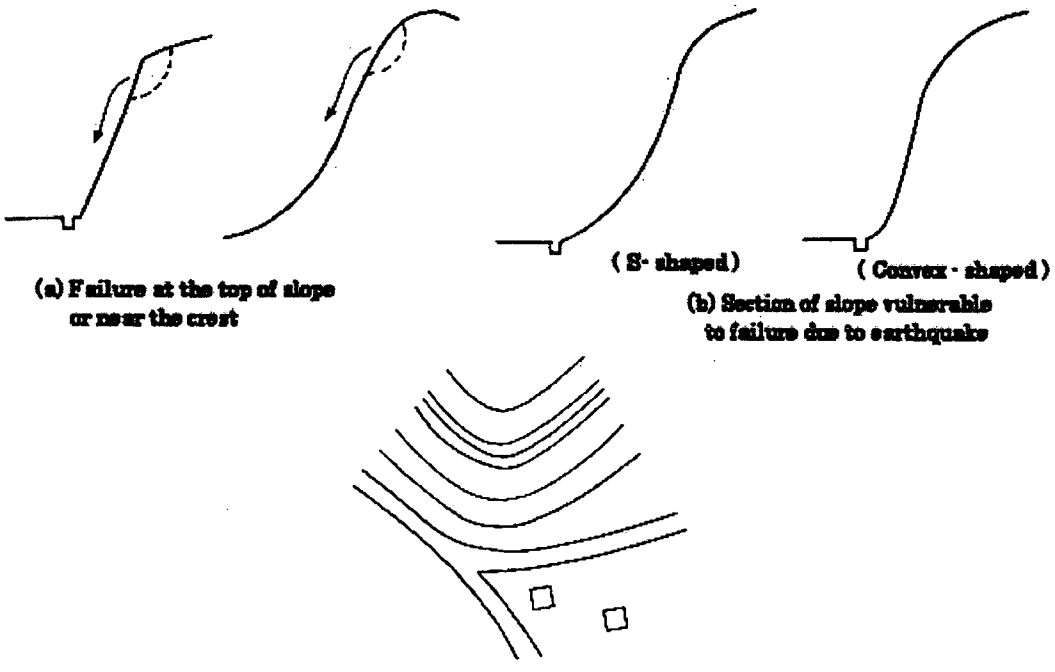


Fig. 2-2-1 Slope Shape Vulnerable to Failure by Earthquake

(8) Rockfalls and Slope Failures from Area above the Cut Slope

In order to prevent the rock fall and slope failure from the area above the cut slope, it is required to investigate the stratum, unembedded and boulder stones with the potential of slope failure or rock fall. The history of such incidence should be understood from a site reconnaissance, topographic map or aerial photographs and appropriate countermeasures to be taken against possible incidence should be studied in advance.

2.2.5 Surveys for Planting

The following surveys should be conducted to meet the preconditions for the completion of planting.

① Observation of the local environment and survey on the surrounding vegetation

When a revegetation target has been established, the possible impacts of planting work and the planned species on nearby plants are examined to achieve continuity and harmony between the slope in question and its surrounding environment.

② Survey on the weather

The temperature, rainfall, wind and sunshine, etc. are surveyed to examine the suitable selection of plants, timing of planting and planting method.

③ Survey on the slope at the time of cutting

(a) Survey on the slope shape and others

The shape, size, height, bearing, gradient, location of spring water, degree of undulation and locations of drainage ditches and structures are surveyed at the slope in question for the selection of suitable plants and the examination of workability.

(b) Investigation on the soil characteristics

The soil hardness, soil texture and soil acidity, etc. are investigated for the selection of suitable plans and the examination of other relevant matters.

④ Utilisation of the survey results

As planting involves live materials, the site conditions significantly affect the success or failure of the work. Important points for utilisation of the survey results for the design and execution of planting are listed in Tables 2-2-3 and 2-2-4.

Table 2-2-3 Gradient and State of Plant Growth

Gradient	State of Plant Growth
Gentler than 1:1.7 (less than 30°)	▪ Restoration of the vegetation dominated by trees is possible. ▪ Local species find it easy to invade from the surrounding area. ▪ Plants grow very well. Once vegetation cover is established, surface erosion virtually disappears.
1:1.7 – 1:1.4 (30° – 35°)	▪ A gradient of less than 35° is required to allow natural invasion from the surrounding area to form plant communities* when the site is left to nature.
1:1.4 – 1:1 (35° – 45°)	▪ Medium high trees and shrubs are dominant and the creation of plant communities where herbaceous plants cover the ground surface is possible.
1:1 – 1:0.8 (45° – 50°)	▪ The creation of low height plant communities consisting of shrubs and herbaceous plants is possible ▪ The introduction of trees may destabilise the foundations in the future.
Steeper than 1:08 (more than 50°)	▪ In principle, slope protection works other than planting are required.

* Plant Community: A group of plants with a virtually uniform appearance and species, such as forest or grassland, and used as a classification unit for vegetation. In the present Guidelines, the planting targets are expressed as such community types as grassland type and shrubland type, etc.

Table 2-2-4 Soil Hardness and State of Plant Growth

Soil Hardness	State of Plant Growth
Less than 10 mm	▪ Germination is poor because of dryness. ▪ The land is vulnerable to failure as the gradient exceeds the angle of repose.
Clayey Soil: 10 – 23 mm Sandy Soil : 10 – 27 mm	▪ Growth of the root system is good (in the case of herbaceous plants, the soil should be fertile). ▪ The soil is suitable for tree planting.
Clayey Soil: 23 – 30 mm Sandy Soil : 27 – 30 mm	▪ Growth of the root system is hampered except for some woody species.
Higher than 30 mm	▪ Growth of the root system is almost impossible.
Soft Rock/Hard Rock	▪ Growth of the root system of woody plants is possible where the rock has cracks.

2.3 Survey on Embankment

2.3.1 Checkpoints of Survey

Surveys of embankment can be roughly divided into foundation ground survey and banking material investigation. The foundation ground should be able to support the weight of the embankment and auxiliary structures without causing any harmful settlement. A careful survey is, therefore, necessary in regard to stability and settlement if the foundation ground is found to be soft. The survey on soft ground is described in more detail in 2.5.

Banking material investigations are conducted to determine the suitability of soil for banking and also to obtain the soil parameters required for the analysis of embankment stability. The test items are as follows.

① Suitability of the soil as a banking material (this is mainly judged from the classification test)
② Strength of the compacted soil (CBR test for the sub-grade, unconfined compressive strength test for evaluation of the stability of a high embankment, etc.)

2.3.2 Slope Failure of Embankment

Type of failure of embankment itself and its slope differ with the condition of foundation ground, the properties of banking materials, scale, shape and execution method of embankment, and rainfall conditions.

Slope failures can be roughly classified shown in Table 2-3-1.

2.3.3 Investigation of the Banking Material

At cutting section or borrow-pit sites, whether or not the required quality and volume of banking material may be acquired should be reviewed and at the same time, the portion in the embankment where the material is to be used should also be reviewed.

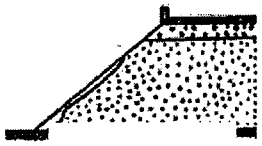
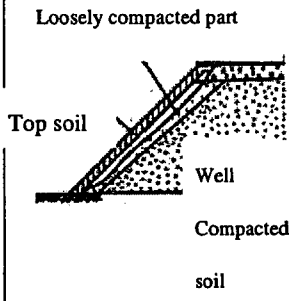
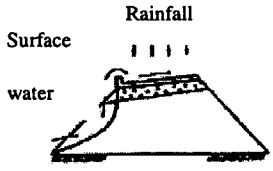
The desirable conditions for banking materials are listed below.

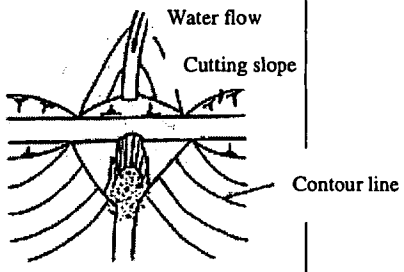
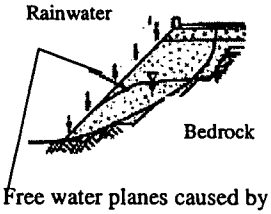
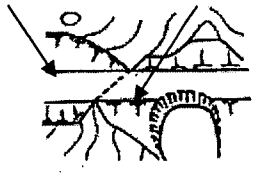
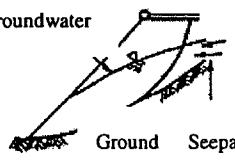
① High density and shear strength to ensure the stability of the embankment
② Ease of compaction
③ Not liable to swelling or shrinkage which damage the stability of the embankment
④ Absence of any organic matter which changes the physical properties of the material
⑤ Little occurrence of pore water pressure during the work
⑥ Trafficability

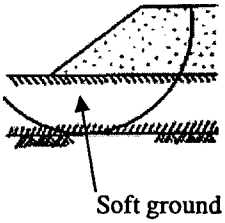
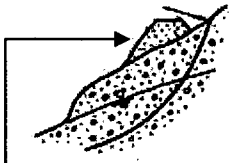
Normally, almost any type of material may be used as banking material but since the swelling property and compressibility of highly organic soil, bentonite, highly alterated rock and heavily weathered serpentinite is high, these materials are not appropriate as banking material. In addition, the following examination is required depending on the type of material to be used.

(i) Cohesive soil with high water content

Table 2-3-1 Classification of Embankment Slope Failure

Category	Description	Schematic Drawing	Typical Geological Features	Remarks
(1) Shallow Failure	1) Surface erosion by rainwater produces a gully. This happens when the slope surface is made of soil liable to erosion, aggravated by insufficient drainage facilities.	 Cross-section  Front elevation	Fine sand; decomposed granite soil;	
	2) Surface failure due to the permeation of rainwater a. Presence of an unevenly compacted part near the slope surface b. Embankment made of a material which has some degree of permeability and of which the strength significantly weakens with a rise of the saturation		Special attention is required for sandy soil and clayey soil, particularly SM, SC and MC, as their strength significantly weakens with a rise of the saturation.	
	3) Scouring and failure caused by the outflow of water on the slope surface, in turn caused by the clogging of drainage works by sediment and grass, etc. a. Sites such as sharply curving sections where the road surface shows a one-way grade, sites where the downward slope in the longitudinal section changes to an upward slope and, particularly, embankment-like sites where the combination of these conditions	 Cross-section	Although scouring or failure has little relation with the banking materials, soil liable to erosion is also liable to failure	

	makes surface water concentrate at a single point b. Clogging of vertical drainage ditches, etc. on the slope surface c. Outflow of stream water on to the road surface due to an insufficient cross-sectional area and/or clogging of a drainage pipe laid across an embankment which crosses a stream.			
(2) Deep Failure	1) Occurrence of an excess pore water pressure in an embankment under rapid construction using clayey soil with a high water content, possibly causing bulging or failure of the slope surface 2) Failure along the boundary between the bedrock and an embankment due to the permeation of rainwater when an embankment is constructed above ground which is steeply inclined in the cross-sectional direction 3) Failure of an embankment due to a strong pore water pressure when an embankment is constructed at a site of which the topography allows an abundant supply of groundwater from the adjacent area a. Embankment near	Excess pore water pressure  Bulging of the slope  Free water planes caused by the permeation of rainwater Cut slope Embankment  Plan Groundwater  Cross-section	Volcanic cohesive soil, such as Kanto loam and others	

	the boundary of cutting and banking sections in either the longitudinal or cross-sectional direction b. Embankment which crosses a stream			
(3) Failure Involving Foundation Ground	1) Occurrence of circular sliding involving the ground when an embankment is constructed above soft ground	 Soft ground	Soft ground	
	2) Occurrence of a major failure as an embankment constructed above talus or at the head of a landslide site assists the further development of a landslide	Talus or head of a landslide site  Embankment	Talus	

Clayey soil with high water content such as volcanic cohesive soil becomes soft when kneaded with construction machinery. As a result, the strength of such materials is low and compressibility is high during construction and immediately after construction. For this reason, such materials are not appropriate for high embankment. In the event high embankment must be constructed using such cohesive soil with high water content, review is required on improving the material through stabilization, admixing with good quality materials or by installing a filtration layer.

(ii) Stones prone to slaking

Among mud stone, shale, tuff and schist, there are weak rocks that show significant slaking or friability. When such materials are used as banking material, these granulate during the construction process or gradually granulate as time passes after construction and cause consolidation settlement over a long period after the embankment is completed. For this reason, slaking test or crushing test of the rocks should be performed to confirm whether or not the material is prone to slaking.

(iii) Sandy soil

In embankment using sandy soil such as fine sand, or decomposed granite soil, since generally the viscosity of these materials is low, the slope may be eroded due to rainfall or slope failure may be generated. For this reason, it is necessary to confirm whether or not the strength of the material changes significantly depending on the water content.

(iv) Silty sand

In the embankment using soil such as silty sand that belongs to the SM or SC category under the Japanese unified soil classification system, high pore water pressure may be generated within the embankment. For this reason, if high embankment are planned using this type of soil, review should be given to installing a filtration layer within the embankment.

(v) Rock mass

In the event material with differing grain size distribution is to be used for the upper layer of an embankment using rock masses, boulders or cobblestone, since there is the possibility that such upper materials will cave in between the rock masses causing differential settlement, review should be given to the installation of an interim layer for adjusting the difference of grain size.

With note made of the above issues, review should be given to performing not only tests for physical properties but also tests for mechanical properties and also to examining examples of construction using the same type of material in order to identify appropriate measures that ought to be taken.

2.3.4 Survey on Embankment Slope Stability

The focal points and survey method for an Embankment are shown in Table 2-3-2. The basic principle for the stability analysis of embankment slopes is to conduct a strength test using a test sample of which the water content and degree of compaction are almost identical to the construction conditions. In this case, the water content of the sample is not the water content of the original ground but is the water content during construction which takes the weather conditions at the time of banking work into consideration. In Japan, it must be noted that the water content during construction is some 5 – 10% higher than the water content of the original ground because of rainfall.

2.3.5 Survey on Embankment Requiring Extra Precautions

The stability of the slope and the main body of embankment is mainly affected by spring water and rainfall, and extra precautions should be taken to the water.

Table 2-3-2 Focal Points of Survey at Embankments and Survey Methods

Main Focal Points of Survey	Survey Method	Soil Test														
		Measurement of Natural Water Content	Liquid and Plastic Limits Test	Specific Gravity Test for Soil	Water Absorption and Bulk Specific Gravity Test	Viscosity Test (Sieve Analysis)	Soil Compaction Test using a Rammer	Cone Index Test (Laboratory)	CBR Test	Unconfined Compression Test	Triaxial Compression Test	Consolidation Test	Frost Heaving Test	Freezing and Thawing Test	Crushing Test for Rock	Slaking Test for Rock
Continuity of Soil Formation and Soil Type		O	O			O										
Stability of Embankment Slope	Clay; Clayey Soil	O	O				2) O		1) O	1) O	1) O	1) O				
	Fine Sand and Sandy Soil, etc.	O	O			O	O				1) O					
Compression of Embankment Itself		O	O	O		O	O					1) O			5) O	5) O
Trafficability of Construction Machinery		O	O			O	O	O								
Use as Backfilling Material for Subgrade	Reference Test	O	O	O	O	O	O		O							
	Long-Term Stability against Weathering and Fragmentation	O	O	O	2) O	O		O							O	O
	Stabilization Test	O	O	O	O	O	O		O	4) O						
	Stability against Frost Heaving and Freezing and Thawing	O	O			O			O	4) O			3) O	3) O		
Criteria and Method for Compaction Control		O		O	O	O	O									

Notes

1) This test is conducted for a compacted specimen by a rammer in a mould.
2) This involves grain size analysis after compaction by a rammer.
3) Special testing apparatus is required.
4) The dimensions of the specimen are 10 mm in diameter and 12.7 cm in height.
5) This test checks the degree of resistance of mudstone, etc. to slaking.

(1) Embankment on Inclined Ground

In case of embankments on inclined ground, embankments in valleys, half-bank and half-cut or transition boundary of cuts and embankments, spring water from ground frequently permeates into embankment, and consequently makes embankment unstable. Therefore, the real conditions of groundwater should be fully investigated particularly. The main items to be clarified with respect to groundwater are:

① Distribution of groundwater, or groundwater pressure
② Extent of permeable layer or aquifer, or extent of impermeable layer
③ Direction of groundwater flow, water vein, water source

These items and groundwater conditions cannot be determined from the results of a single survey and should thus be comprehensively determined from the results of many surveys including site reconnaissance, borings, sounding and so forth.

Also, seasonal changes in groundwater are often remarkable in many cases and it is therefore desirable to understand these changes.

(2) Presence of Soft Ground

At paddy fields and wetland, the soft surface layer tends to be found. This soft layer has the problems described below if it is to act as the foundation ground for an embankment.

It is difficult to secure trafficability during the construction of the first layer of an embankment. In the case of a low embankment, the traffic load after completion could affect the soft layer, causing deformation of the road surface and the eventual destruction of the pavement. Drainage of the foundation ground as shown in Fig. 2-3-1 is important in this sense.

Given the likelihood of the above problems with paddy fields and wetland, the thickness and distribution of the soft layer should be checked by means of the Swedish weight sounding test, the Dutch cone penetration test or similar.

(3) Embankment at Landslide Site

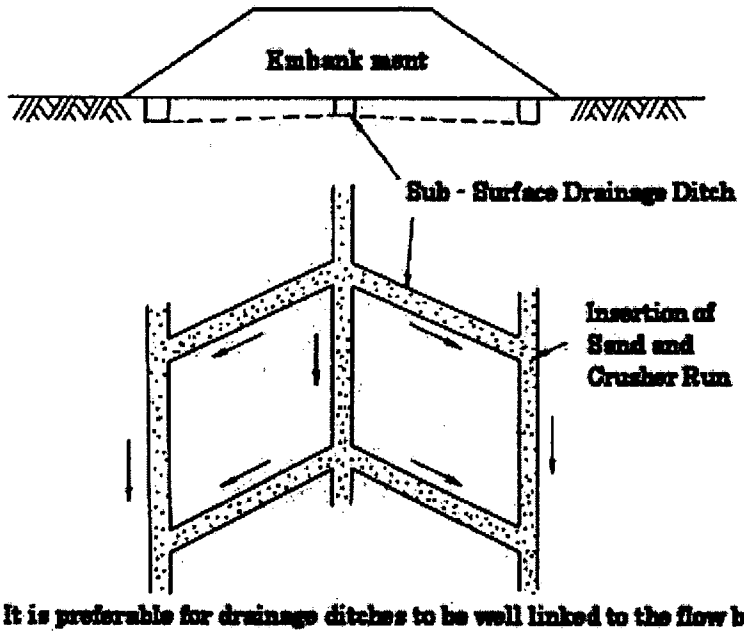


Fig. 2-3-1 Drainage from Foundation Ground for Embankment at Paddy Field or Swamp

The construction of an embankment at a landslide site is extremely dangerous and huge cost is incurred by the countermeasures required to ensure its stability.

As landslides tend to occur at sites with special geology and topography to a certain extent, the same or a similar type of landslide is likely to occur in areas with similar geology and topography. Accordingly, it is recommended to conduct survey based on documented records of the local geology and topography, records of landslides in neighbouring areas and the memories of local elders (see Fig. 2-3-2).

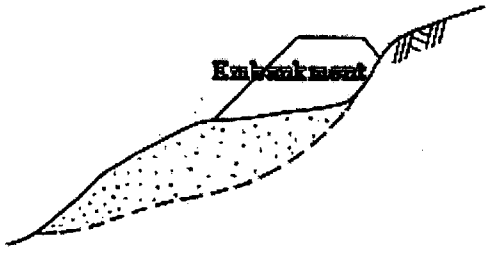


Fig. 2-3-2 Embankment at Former Landslide Site

(4) Embankment prone to Damage by Earthquake

Viewing past damages and conditions under which such damage is prone to occur as a result of earthquakes, cracks, settlement and failure occur when the foundation is weak or the level of the ground water is high causing liquefaction of loose sandy soil, when the dip of the ground is steep or at the location of transition boundary of cuts and embankment in mountainous regions, when the water drainage facility is faulty at locations that are prone to concentrating seepage water at all times or when the water content in the

embankment is high due to rainfall prior to the earthquake.

Such factors as the height of the embankment, the gradient of the slope, the banking material, the type of the stratum, the dip of the ground, the N value of the foundation, the status of water catchment and the type of existing preventive measures impact stability and extent of damage of an embankment in the event of an earthquake. Accordingly, it is necessary to review the stability of an embankment during an earthquake in the event that the impact of failure is expected to be significant. In such cases, the condition of the embankment and the foundation ground need to be investigated through boring, sounding and laboratory testing of the soil.

2.4 Survey on Drainage Systems

2.4.1 Checkpoints of survey

A drainage facility may be for the purpose of ensuring the stability of the earth structure such as an embankment or ensuring the quality of subgrade and base course, for the purpose of ensuring the efficient construction work or for the purpose of ensuring smooth traffic.

A survey on drainage facilities is undertaken with the above checkpoints taken into consideration and on the items shown in Table 2-4-1. The following points need to be noted in performing the actual survey.

① Locations where the surface water flows in a locally concentrated manner
② Locations with a large amount of spring water or seepage water
③ Ground water condition
④ Condition of downstream for discharge of the collected water

Since the stratum structure of the foundation ground, soil properties and other such factors relate to the flow of seepage water in the ground in a complex manner, it is difficult to ensure accurate understanding through only the preliminary survey. For this reason, in many cases the existence of ground water or seepage water is discovered during the construction work. Therefore, it is important always to observe surface water and seepage water during the construction work.

Moreover, care is also required as the level of the ground water may decrease thus impacting the ground water condition in the vicinity as a result of cutting.

Table 2-4-1 Survey for Drainage System

	Item of Survey	Purpose of Survey
1	Meteorology	Determination of run-off Design for drainage during execution
2	Topography and the coverage of ground surface	Determination of run-off Groundwater flow calculation
3	Soil and geological structure, groundwater etc.	Design for drainage during execution Determination of groundwater drainage Determination of slope drainage
4	The condition and drainage capacity of the existing drainage canals, which belong to the same drainage area and the drainage system, and the manager of canals	Determination of run-off Plan of new drainage system

2.4.2 Survey on Drainage System for Surface Water

In the case of drainage facilities that target surface water as a result of rainfall or water that flows from the vicinity area, the major factors to be investigated are the meteorology and the catchment area. In investigating the meteorology, records at least last 10 years should be collected from the meteorological observatory or reference should be made to other construction work undertaken in the vicinity of the construction site. With respect to the catchment area, the area should be calculated by the each type of surface for both the position of the construction site and adjacent area. In particular, since mountainous areas have numerous slopes and in many cases the catchment area is not clear, it is desirable that aerial photographs be used to determine the catchment area.

In the case of drainage on slope surface, as the objective is to prevent erosion and scouring of the slope surface due to the flow of surface water, in addition to materials for top soil, adequate survey of the topography, the surface condition, soil properties, ground water condition, existing drainage systems and other such factors is required. In particular, locations that are prone to concentration of surface water or in other words, valleys, hollow, remains of cave-ins, remains of small scale landslides, places that will become large slope and places that will involve sharply curved cant embankments need to be investigated carefully. Moreover, slope surfaces comprised primarily of sandy soil, such as fine sand, decomposed granite soil or terrace sand and gravel are prone to erosion by surface water and particular care is required.

2.4.3 Survey on Drainage System for Seepage Water

In planning subsurface drainage facilities targeting seepage water, the stratum structure of the foundation ground and the ground water condition should be carefully investigated in consideration of the road structure.

First, existing data should be collected, aerial photographs utilized, site reconnaissance undertaken, hearings

implemented to obtain comprehensive information on the ground water at the planned site. Based on such information, in consideration of the ground water system and water balance whether or not the ground water will act as an impediment should be reviewed. When the topography is divided into such categories as plateau, alluvial cone, hill, sand dune or lowland, since ground water has specific properties for each type of topography, these guidelines should be referenced to undertake preliminary survey. Moreover, in general, where ground water seeps to the surface, there is a gravel bed or fractured zone above the impermeable stratum and plants on the surface differ from those in the vicinity. These points should be carefully noted during site reconnaissance towards discovery of spring water and wetlands.

If lowering the ground water level or draining the ground water is deemed necessary as a result of the preliminary survey, the main survey should be undertaken.

In the main survey, detailed review is made on the stratum composition of the ground and the ground water condition through identifying changes in the water quality and volume at typical wells and springs, measuring the water flow at major rivers and other water channels, investigating geological stratum through boring at typical locations, electric prospecting and in-situ permeability test in accordance with the situation of the site. In particular, slopes consisting of talus materials, faults, fractured zone, and stratum combining hard and soft soil often have highly permeable strata such as those composed of gravel or sand bed. These strata comprise aquifer providing seepage water or ground water and thus generating failure of the slope surface. For this reason, adequate survey of the composition of the various strata, permeability and changes in the ground water condition is required.

2.5 Survey on Soft Ground

2.5.1 Checkpoints of Survey

Soft ground is comprised of soft soil with large content of very fine particles such as clay or silt, organic soil with large pores such as peat, or loose sand. In general, the level of the ground water is high and such foundation ground settles or causes problems in stability as a result of the load of embankments or structures.

The stratification and soil properties of soft ground normally differ widely by the environment of generation depending on the topography. When categorizing the location of soft ground from a topographic perspective, the 6 rough groups are branched valley, back marsh, small-scale drowned valley, flat delta, coastal reclaimed ground and areas along natural levee and coastal bar. Typical stratification and properties of ground at each of these areas are shown in Table 2-5-1. Moreover, Table 2-5-1 shows typical value of natural water content w_n, natural void

ratio e_n, unconfined compressive strength q_u and the N value obtained from the standard penetration test categorized into peaty ground, clayey ground and sandy ground, which are observed in the past construction work. Accordingly, with this type of ground, review should be made as possible soft ground.

Table 2-5-1 Classification of Soft Ground and Typical Soil Property

Topographic Zone	Ground classification	Soil classification			Symbol	Soil property			
						ωn (%)	e_n	q_u(kgf/cm^2)	N-Value
Branched valley Back swamp marsh	Peaty ground	Highly organic soil Pt	Peat (Pt)	Fibrous highly organic soil		Above 300	Above 7.5	Under 0.4	Under 1
			Muck (Mk)	Decomposed highly organic soil		300 to 200	7.5 to 5		
Small-scale drowned valley Flat delta Coastal reclaimed ground	Clayey ground	Fine-grained soil F	Organic soil {O}	Under A line of plasticity chart, organic soil		200 to 100	5 to 2.5	Under 1	Under 4
			Volcanic cohesive soil {v}	Under A line of plasticity chart, volcanic secondary sedimentation cohesive soil					
			Silt {M}	Under A line of plasticity chart, large dilatancy		100 to 50	2.5 to 1.25		
			Cohesive soil {C}	Above or around A line of plasticity chart, small dilatancy					
Natural levee coastal bar	Sandy ground	Sand particulate soil S	Sandy soil {SF}	Grain size 75 μ or below : 15 to 50 %		50 to 30	1.25 to 0.8	----	10 to under 15
			Sand {S}	Grain size 75 μ or below : under 15 %		Under 30	Under 0.8		

When constructing a road on soft ground, there is the danger not only of damage to the road itself due to instability of the embankment or excessive settlement but also of damage in adjacent area of the road. For this reason, thorough examination in the design and construction of road structure is necessary, and the soil investigation needs to be particularly detailed.

The objective of soil investigation of soft ground is to elucidate the extent of soft ground (depth and extent), composition of soil stratum and its engineering properties (shear strength, rate of strength increase, characteristics of consolidation, unit weight, water content, organic matter content, etc.) of the various strata in order to examine the stability and settlement of ground due to the load of embankment, and to select the countermeasures.

Concretely, sounding and boring of the soft ground is undertaken and samples of undisturbed soil are taken as necessary to perform soil tests and in situ tests as part of the investigation.

The following are points that need to be noted in investigating soft ground.

(i) When focus is placed particularly on soft ground, the procedures for road construction are shown in Fig.2-5-1. In performing the survey, these procedures on soft ground should be kept in mind and methods, scale and detailed conditions of the survey that are in conformance with the accuracy requirements of the various stages should be determined.

(ii) In general, based on the existing data that are collected and the results of preliminary survey with particular emphasis on site reconnaissance, sounding, boring, sampling and soil tests are implemented. The greater the volume of pertinent information collected during the preliminary survey stage, the less wasteful will be the later survey and the more appropriate survey plan can be made. For this reason, preliminary survey is indispensable.

(iii) At the stage of the main survey, a detailed survey with respect to stability and settlement is required particularly in the following condition and supplementary survey is performed as required.

① When the required safety ratio is not obtained through approximate calculations on stability;
② When a significantly large settlement is expected from approximate calculations or when secondary consolidation settlement that continues over a long period is expected;
③ When the change in the soft stratum in the longitudinal and transverse direction of the route is significant such as in a narrow valley;
④ When the foundation ground forms complex alternated strata or when there is a stratum with large consolidation;
⑤ When there is a stratum that is particularly weak or significant consolidation is expected;
⑥ When it is expected that the settlement or sliding failure of the embankment will seriously impact the vicinity area;
⑦ When the foundation ground consists of sand with possibility of liquefaction.

(iv) The properties of soft ground are complex and not uniform and normally involve significant changes of stratification and soil property in both the plane direction and depth direction. For this reason, attempting to judge the overall situation from the results of a single survey may lead to inappropriate conclusions. Accordingly, it is important to judge the results of the survey from a comprehensive perspective and towards this, each factor should be clearly linked and the appropriateness of the various values should be judged in consideration of the interrelationship among the results of the soil tests obtained at the various

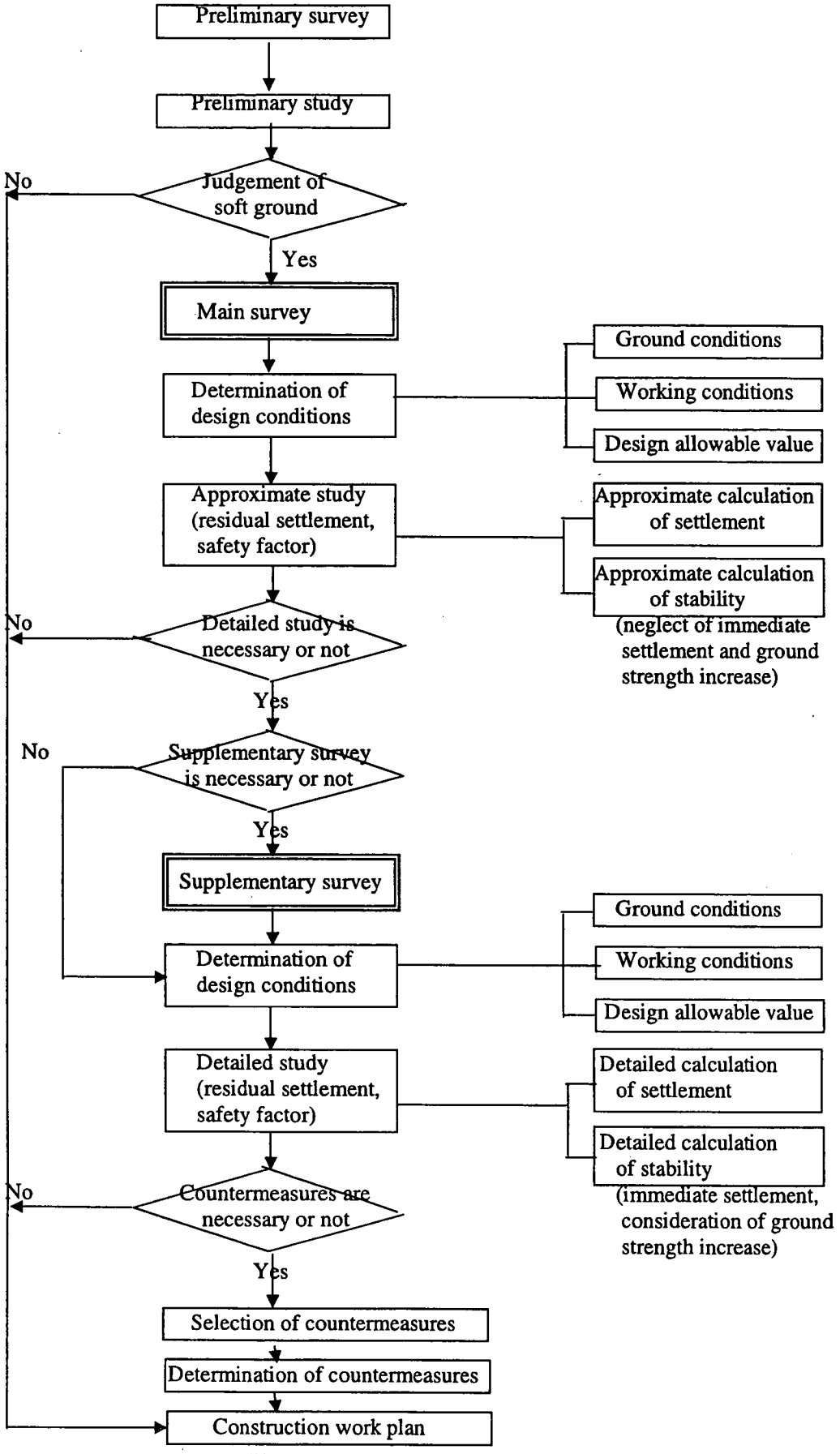


Fig. 2-5-1 Procedure for Study on Soft Ground

points investigated. The various values for the foundation ground that need to be used in the design should be selected among these values.

(v) Prior to the construction work, real scale test banking should be undertaken and settlement and stability should be measured using settlement plates, displacement stakes, in-situ strain meter, piezometer and other such devices. The method of comparing theoretically estimated values to actually measured values is often used in the survey on soft ground.

2.5.2 Survey on Settlement and Stability

The following is the standard concept concerning the objectives, scale and detailed conditions of each survey.

(1) Sounding

In order to investigate the condition of the stratification of soft ground, such methods of sounding as Swedish weight sounding, Dutch cone penetration test and standard penetration test are appropriate. Moreover, these sounding methods allow estimation of the approximate value of soil modulus from the penetration resistance.

The depth of investigation in sounding is, in principle, up to a point below the soft ground where solid foundation ground is confirmed and the interval between points investigated shall be every 20 to 100 meters along the center line of the road depending on the extent of change in the topography or soil stratum. Where the changes in the topography are large or where the soft ground stratification in the transverse direction of the road is complex, sounding should be undertaken on both sides of the right-of way as well in order to elucidate the changes in the soil stratum in the transverse direction.

(2) Sampling

In order to investigate soil modulus in detail, it is necessary to collect undisturbed samples using the thin walled sampler or foil sampler and to perform various soil tests of such samples.

The locations for sampling should be determined using the results of the sounding and boring as reference and points at which the soft stratum is thick or the strength is particularly weak should be selected. In principle, the sampling points shall be along the center line of the planned road. The standard intervals for sampling along the longitudinal direction of the road shall be 50 to 100 meters but in any event, sampling shall be undertaken at minimum 2 points in each sections of soft ground. Moreover, the depth of the

sampling shall in principle reach below the soft ground to the bearing stratum. The interval in the depth direction of sampling utilizing the thin walled sampler shall ensure that samples are collected from at least 1 point of soil stratum that is deemed to be uniform and if the thickness of the stratum is large, it is desirable to collect samples from 2 to 3 points of the same soil stratum. Moreover, while this may seem standardized, it is practical to undertake sampling using the thin walled sampler at intervals of 1 to 2 meters in the depth direction.

(3) In Situ Test

The vane test may be used as a test directly to obtain the shear strength of soft ground in situ. This test is particularly effective for fibrous organic soil of which undisturbed sampling and soil tests are difficult and with significant anisotropy in strength.

(4) Soil Test

Values from soil tests that need to be obtained for use as reference in categorizing the soil stratum of soft ground are natural water content, specific gravity of soil particle, liquid limit, plastic limit, grain size distribution and organic matter content (in the case of peaty ground). It is desirable to obtain continuous natural water content distribution in the depth direction and other values may be obtained at the representative point in the soil stratum.

As shear strength for the stability analysis of embankment, the general method is to obtain the unconfined compressive strength considering stability during the rapid banking. In the event design taking into account the increase of strength of foundation ground is to be undertaken, in other words if staged construction is planned, consolidated undrained triaxial compression test should be performed.

The unconfined compression test should be undertaken with 2 to 4 samples per single thin walled tube or per meter of foil sampler. The triaxial compression test requires at least 4 specimens to obtain the strength parameters c_{cu}, ϕ_{cu} or c', ϕ' and in principle the test involves changing the consolidation pressure for the 4 specimens.

Next, consolidation test needs to be performed to obtain the consolidation properties of the soil in order to calculate the settlement. The number of samples required is between 2 to 3 specimens per single thin walled tube or per meter of foil sampler.

(5) Arrangement of the Survey Results

The results of survey on soft ground obtained from the above tests should be compiled for the depth direction for the sounding, in site test, boring log, soil test and other such tests by each point and produce a soil test overview diagram. Moreover, based on these results, a soil profile needs to be produced and compiled together with a plane figure on which the survey points are marked. Examples of a soil test overview diagram and soil profile are shown in Fig. 2-5-2 and Fig. 2-5-3 respectively.

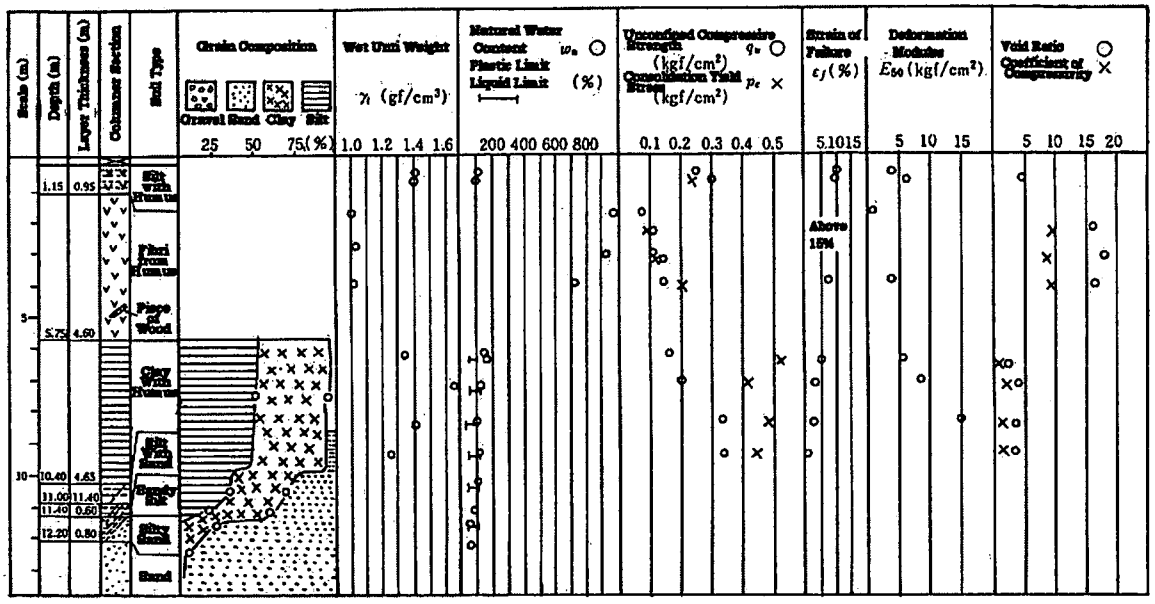


Fig. 2-5-2 Example of Chart for Soil Test Results

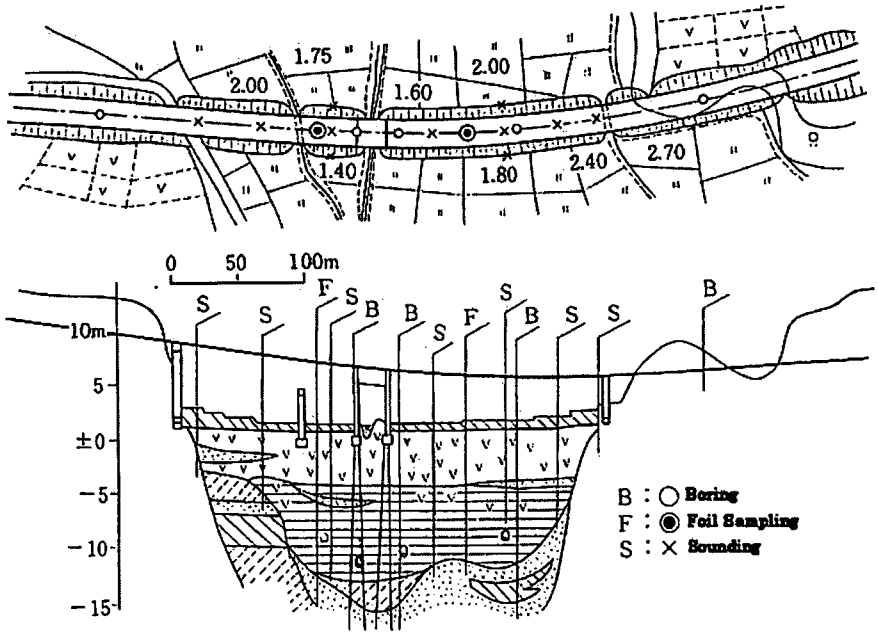


Fig. 2-5-3 Example of Soil Profile

2.5.3 Survey for Judgement on Liquefaction

Judgement on liquefaction is usually based on the simplified method using the liquefaction resistance factor of F_L. This simplified method requires the various parameters listed below. As the values of these parameters frequently vary depending on the horizontal positions as well as relative positions to the depth direction even in strata which are judged to be uniform, an investigation with a relatively high frequency is required.

① Geological composition; groundwater level
② Unit weight of the ground above the groundwater level
③ Unit weight of the ground below the groundwater level
④ N value (as determined by the standard penetration test)
⑤ Plasticity index (as determined by the consistency test)
⑥ Fine-fraction content (as determined by grain size analysis)

(1) Sandy Soil Stratum in Need of Judgement on Liquefaction

A saturated sandy soil stratum of alluvium to which all of the following three conditions apply has a prospect of liquefaction at the time of an earthquake and, therefore, judgement on liquefaction must, in principle, be made in accordance with the method described in (2) below.

① Saturated stratum within 20 m of the original ground surface when the groundwater level is within 10 m of the said ground surface

② Stratum of which the fine-fraction content F_c (passing percent weight of soil particles with a size of 75 μm or less) is 35% or less or stratum of which the plasticity index I_p is 15 or less even if the F_c is above 35%

③ Stratum of which the mean particle size D_{50} is 10 mm or less and of which the 10% particle size D_{10} is 1 mm or less

(2) Liquefaction Judgement Method

Using Expression (2.5.1), the liquefaction resistance factor F_L is calculated for strata in need of judgement on the liquefaction prospect based on (1) above and strata of which the value of F_L is 1.0 or lower are considered to become liquefied.

$$F_L = R / L \quad \text{..... (2.5.1)}$$

$$R = c_w \cdot R_L \quad \text{..... (2.5.2)}$$

$$L = r_d \cdot k_s \cdot o_v / \sigma_v' \quad \text{..... (2.5.3)}$$

$$r_d = 1.0 - 0.015x \quad \text{..... (2.5.4)}$$

$$\sigma_v = \gamma_{t1} h_w + \gamma_{t2}(x - h_w) \quad \text{..... (2.5.5)}$$

$$\sigma_v' = \gamma_{t1} h_w + \gamma'_{t2}(x - h_w) \quad \text{..... (2.5.6)}$$

(In cases of medium-scale earthquake motion and interplate-type large-scale earthquake motion)

$$c_w = 1.0 \quad \text{..... (2.5.7)}$$

(In case of direct hit-type large-scale inland earthquake motion)

$$c_w = \begin{cases} 1.0 & (R_L \le 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \le 0.4) \\ 2.0 & (0.4 < R_L) \end{cases} \quad \text{..... (2.5.8)}$$

Where,

F_L : liquefaction resistance factor

R : dynamic shear strength ratio

L : seismic shearing stress ratio

c_w : correction factor by earthquake motion characteristic

R_L : cyclic triaxial strength ratio

r_d : reduction coefficient of seismic shearing stress ratio along the depth direction

k_s : seismic intensity for judgement on liquefaction

σ_v : total top load (kN/m^3)

σ_v' : effective top load (kN/m^3)

x : depth from the ground surface (m)

γ_{t1} : unit weight of the soil above the groundwater level (kN/m^3)

γ_{t2} : unit weight of the soil below the groundwater level (kN/m^3)

γ'_{t2} : effective unit weight of the soil below the groundwater level (kN/m^3)

h_w : depth of the groundwater level (m)

The cyclic triaxial strength ratio R_L is calculated by Expression (2.5.9).

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6\times10^{-6}\cdot(N_a - 14)^{4.5} & (14 \le N_a) \end{cases} \quad \text{..... (2.5.9)}$$

Where

(In case of sandy soil)

$$N_a = c_1 N_1 + c_2 \quad \text{..... (2.5.10)}$$

$$N_1 = 170N/(\sigma'_v + 70) \quad \text{..... (2.5.11)}$$

$$c_1 = \begin{cases} 1 & (0\% \le F_c < 10\%) \\ (F_c + 40)/50 & (10\% \le F_c < 60\%) \\ F_c/20 - 1 & (60\% \le F_c) \end{cases} \quad \text{..... (2.5.12)}$$

$$c_2 = \begin{cases} 0 & (0\% \le F_c < 10\%) \\ (F_c - 10)/18 & (10\% \le F_c) \end{cases} \quad \text{..... (2.5.13)}$$

(In case of gravelly soil)

$$N_a = \{1 - 0.36\log_{10}(D_{50}/2)\}N_1 \quad \text{..... (2.5.14)}$$

Where,

N : N-value obtained by the standard penetration test

N_1 : N-value converted to the equivalent of the effective top load of 100 kN/m^3

N_a : corrected N-value having considered the impact of the particle size

c_1, c_2 : correction coefficient for N value based on the fine-fraction content

2.6 Survey on Rockfall

2.6.1 Checkpoints of Survey

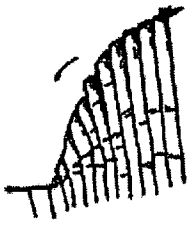
The types of rockfalls are, as shown in Table. 2-6-1, uprooting type (boulder type) and flaking type. These differ in the mechanism of occurrence and this categorization is fundamental to evaluating the possibility of rockfalls or planning countermeasures for rockfalls.

Survey on rockfalls needs to be divided into survey for planning and design and survey for undertaking countermeasures and implemented taking note of the following points.

① Possibility of rock fall (frequency of occurrence, weather conditions, etc.);

② Extent and type of rock fall (size and location of objects that portend rock falls, categorization into the

Table 2-6-1 Classification of Rock Fall

Category	Description	Schematic Drawing	Typical Geological Features	Remarks
(1) Uprooting Type	1) Dropping of gravel located on a sandy slope containing gravel		Terrace and pyroclastic material, etc.	
	2) Sliding down of gravel contained in the sediment above the bedrock		Heavily weathered rock on talus, colluvial soil, hillside slope or cut slope	Loose stone type rock fall may occur depending on the relative positions of the bedrock and sediment.
(2) Flaking Type	1) Exfoliation along the surface of discontinuity in the bedrock		Bedrock with many cracks or continual cracks	Different forms, such as sliding, falling over and falling, take place depending on the directions of the slope and cracks; special attention is required in regard to the degree of looseness.
	2) Exfoliation of the surface of a highly weatherable or erodsible bedrock		Weatherable soft rock or alternation of soft and hard rock layers from the Neogene onwards	The scale is generally small but the falling of large rocks may occur at an overhang section of alternated soft and hard rock layers.
(3) Others	1) Increased instability of residual boulders, etc. on a ridge due to weathering or erosion		Weathered granite, etc.	Not frequent but tends to be on a large scale.

uprooting type or flaking type);

③ Dynamics and path of the rock fall (sliding, rolling, jumping, etc.);

④ Margin at the sides of the road;

⑤ Extent and range of rock falls reaching the road;

⑥ Condition of the ground at locations where preventive countermeasures need to be undertaken;

Based on the above points, surveys shall be undertaken using the following methods and on the following factors.

① Survey of existing materials: Survey of records of past rock falls, survey of the weather upon occurrence of rock falls in the past;

② Interpretation of aerial photographs: Size and location of objects that portend rock falls, condition of the path of rock falls, positional relationship of slopes where rocks falls occur and the road, undulation of the slope, vegetation, etc.;

③ Site reconnaissance: Size and position of objects that portend rock falls, possibility, status of plants, detailed topography of slopes, etc.;

④ Elastic wave exploration, boring, sounding, etc.: Degree of weathering of the slope, degree of fissures in the slope, position of basement rocks, strength of ground, etc.;

2.6.2 Survey on Rockfall Requiring Extra Precautions

(1) Steep Slope with Protrusions in the Cross Section Profile

Slopes that have protrusions in the cross section profile (perpendicular to the contour line) or slopes that have protrusions in the plane section may result in fallen rocks hitting the protrusions and rising to an abnormal height and the topology of such slopes need to be investigated in detail.

In particular, slopes with overhangs are prone to rock falls and care needs to be taken with detailed survey of the geological conditions.

(2) Steep Valley Shaped Slopes adjacent to the Road or High Natural Slope above Cut Slope

When rocks fall on the surface of slopes, the plane tracks are macroscopically perpendicular to the contour lines. Accordingly, at slopes with protrusions (ridge shaped) as shown by the arrows in Fig. 2-6-1 are prone to falling in various directions in a dispersion while at slopes with indentations (valley type), the rocks have possibility to concentrate along the valley lines. In case of steep and high slope as shown in Fig. 2-6-1 (c), the probability of the occurrence of rock falls is high and when such rock falls occur, there is the high possibility of the rocks with high energy hitting road facilities. For this reason, care needs to be taken in this kind of slope.

(3) Slopes with Rock Types and Rock Properties that Require Special Care

Among rock types and rock properties that comprise slope surface or slopes, particular care is necessary for

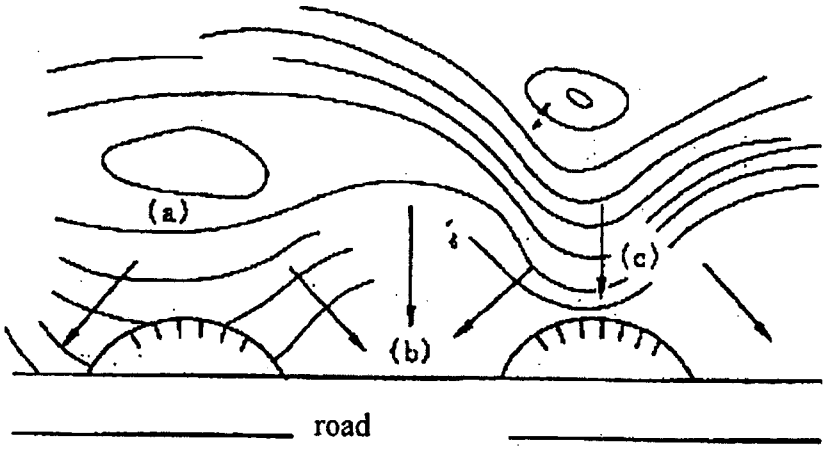


Fig. 2-6-1 Plane shape of slope and falling route of rock

granite, schist, pulverized slate, alterated stratum with mud stone and sandstone, lava and talus materials. Moreover, highly welded tuff or old serpentinite and tuff generate regular fissures that can cause rock falls.

(4) Slope with Histories of Disaster due to Rockfalls

Rockfalls tend to occur in relatively same place in succession and it is said that such a phenomenon becomes frequent. For this reason, past damage to slopes and in particular slopes that have a history of rockfall occurring needs to be investigated as to what the weather conditions were and how the rockfall was generated.

(5) Slopes with regards which Time Dependency Change in Local Shape is Identified

With respect to slopes with regards which localized change in shape with the passage of time is identified, such as slopes with regards which the probability of uprooting type rockfalls is increasing with progressing exposure of rock masses, boulder or gravels through matrix erosion or the probability of flaking type rockfall is increasing due to enlargement or loosing of rock fissures causing instability in rock blocks surrounded by fissures, particularly detailed survey needs to be undertaken. In such cases, the current state of the change and its forecast progress need to be estimated to evaluate the probability.

2.7 Survey on Landslide

2.7.1 Checkpoints of Survey

Landslides tend to occur frequently in areas with specific geology or geological formations such as Tertiary mudstone or tuff zones or Mesozoic and Paleozoic metamorphic rock zones, and such areas frequently indicates unique topographic features due to past successive landslide activity. The relation between the topographical features of landslides and the properties of sliding mass in the process of landslide movement have been generally recognized and are indicated in Table 2-7-1. Landslides are also frequently induced by small-scale earthworks in areas where the topographical features of landslides are apparent. Such landslide zones can be frequently determined by aerial photographs.

A survey on landslides is conducted to obtain the following types of information when a road is planned at a landslide site or at a site where the possibility of a landslide is recognised.

① Extent and scale of sliding
② Direction and velocity of sliding and presence of a scarp
③ Stability of a landslide and changes of the safety factor due to earthwork and other causes
④ Mutual relationship between a landslide and contributory factors
⑤ Location of installed measuring instruments

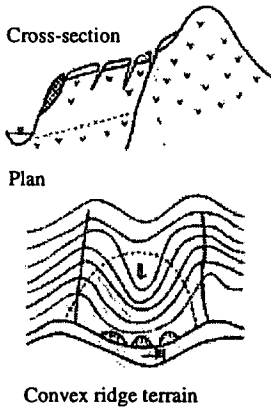
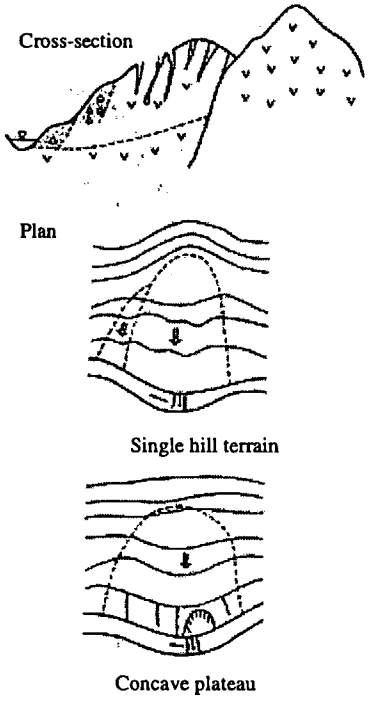
The entire picture of a landslide is clarified based on the above information and a suitable route is selected or adequate countermeasures are implemented by investigating the causes and mechanism of the landslide.

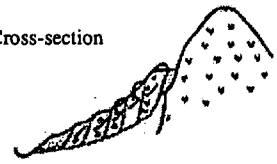
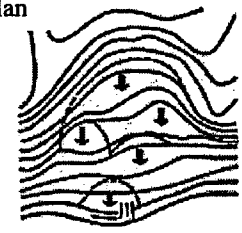
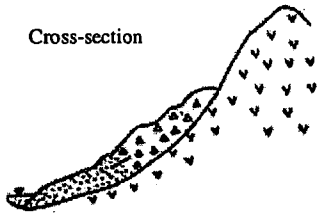
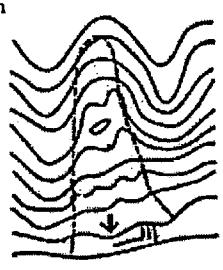
There will be a high potential of landslide occurrence requiring detailed surveys if the following symptoms are found in the survey areas after reading the topographic maps and aerial photographs.

① Irregular arrangement of contour lines
② Presence of horse-shoe shaped type cliffs or plateau type slopes, or presence of dense contour lines below gentle slopes
③ Regularly arranged ponds, swamps and marshes
④ Presence of irregular contour lines in front of steep cliffs, or presence of separate small hills
⑤ Abnormal curves of such as gorges, streams, abnormal curves and changes in the grade of road
⑥ Presence of terrace paddy field, debris flow deposit

2.7.2 Survey on Stability of Landslide

Table 2-7-1 Classification of Landslides

Category	Description	Schematic Drawing	Typical Geological Features	Remarks
(1) Bedrock Landslide	1) Convex ridge terrain with a chair or boat-type sliding surface; often starts at a saddle section 2) Bedrock or lightly weathered at the head and weathered rock at the bottom 3) Sudden occurrence makes its prediction very difficult; careful reconnaissance and a detailed survey are required		Often affected by a fault or fracture zone; Tertiary formations, crystalline schist and Mesozoic and Paleozoic formations	Triggered by large-scale earthworks, submersion of part of the slope, earthquake or heavy rain
(2)Weathered Rock Landslide	1) Convex plateau terrain, single hill, concave plateau terrain with a chair or boat-type sliding surface 2) Weathered rock with many cracks at the head and sediment mixed with boulders at the bottom		Crystalline schist, Mesozoic/Paleozoic formations or Neogene formation affected by a fault or fracture zone	Triggered by a downpour, abnormal thawing, earthquake or medium-scale earthworks
(3) Colluvial Deposit Landslide	1) Multiple hill or concave plateau terrain with a stair or layer-type sliding surface which can be divided into 2 – 3 blocks 2) Mainly consists of sediment		Colluvial deposit from crystalline schist, Mesozoic/Paleozoic formations, Neogene formation or serpentinite	Triggered by thawing, typhoon, downpour or medium-scale earthworks

	containing gravel and becomes clay at the bottom 3) Intermittent activity repeated every 5 – 20 years; landslide hysteresis is clear from the topographical point of view and can be checked with a topographical map of 1 to 5,000 or 1 to 10,000; interviews with local people are also useful	Cross-section  Plan  Multiple hill terrain		
(4) Clayey Soil Landslide	1) Gently sloping concave terrain with a multi-blocked stair or layer type sliding surface; closer relation between the blocks than (3) 2) Mainly consists of clay or clay containing gravel 3) As landslide movement is semi-continuous with a recurrence every 1 – 5 years, its presence is well-known locally.	Cross-section  Plan  Gently sloping concave terrain	Neogene formation, fracture zone and solfataric soil	Easily activated by a downpour, thawing, river erosion or small-scale earthworks

Surveys are mainly performed by borings and sometimes the excavation of test pits is also performed to study the landslide mechanism from the viewpoint of soil mechanics and to examine the soil mechanical characteristics in detail of sliding surface clay.

For soil and geological survey, at least four borings should be made and three of these should be on and across the sliding block and one should be on the upper slope of a block. It is important to plan and actually make a boring at least 5 m into the bedrock in order to distinguish the sliding mass from the bedrock. Also, where the landslide area

is wide and the distribution of bedrock is quite irregular, it is also important to perform elastic wave exploration and to provide an auxiliary course of traverse to allow a review in the traverse direction.

From the mechanical point of view, this survey is performed to determine the hardness of the landslide mass and the soil texture, and also to determine the strength of soil mass required for the stability analysis of the slope in the landslide area and for designing countermeasures with the results of sounding, standard penetration tests, in-situ tests, and tests for physical and mechanical properties.

The survey results are then compared to the results of the field survey and topographic and geologic map readings, and then the characteristics and extent of landslide activity are examined.

2.7.3 Survey on Contributory Factors of Landslide

It is well known that landslides frequently occur during periods of heavy rain and that landslide movement becomes active as the groundwater level rises. Information concerning the mechanism and possibility of landslides can be obtained by determining the groundwater conditions in the landslide area (fluctuation in ground water level, flow of groundwater, runoff path, current speed, quality and temperature of groundwater, etc.) to make it possible to examine the quantity and location of effective drainage work as countermeasures based upon the state of groundwater. Groundwater surveys can be generally classified into groundwater distribution surveys and groundwater pressure surveys. The contents of the survey are as shown in Fig. 2-7-1, and the scope of survey should be determined according to the purpose.

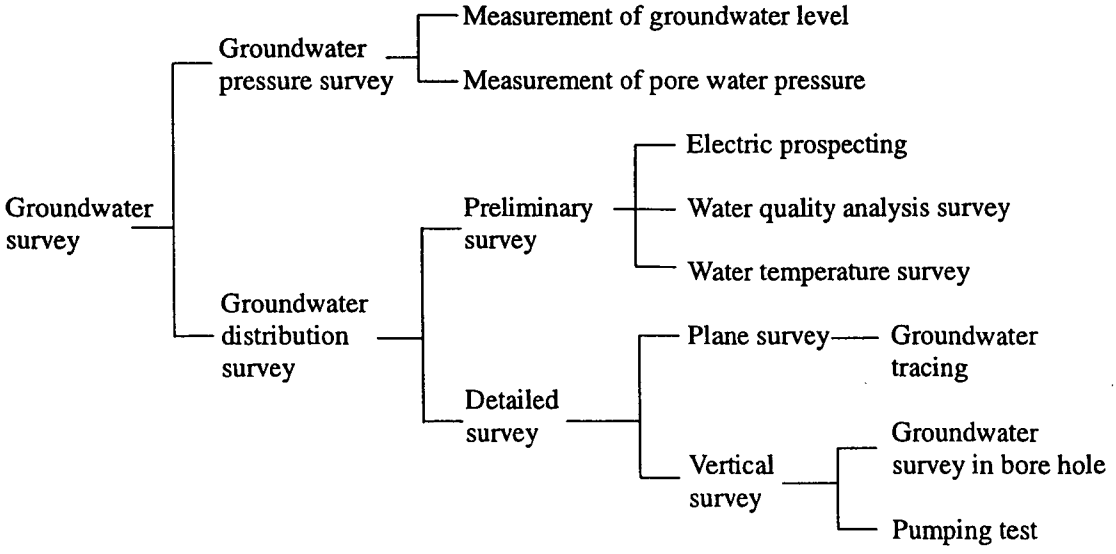


Fig. 2-7-1 Groundwater Survey

Groundwater should be surveyed by continuous observation since it often has seasonal fluctuations. In addition, vegetation in the landslide area is closely related to the distribution of the groundwater zone and, this, must be compared to the results of field survey.

2.7.4 Survey on Extent and Movement of Landslide

This survey is performed to examine the extent, direction of movement, and the mechanism of landslides in detail when any sign of landslide motion such as slide scarps or cracks are found or when there is any possibility of the occurrence of landslides in the future.

Measurement or monitoring survey may involve monitoring of movement or survey on sliding surface. Monitoring of movement is undertaken in order to confirm the extent of sliding and to obtain material for forecasting future activation of landslides and measures the amount of fluctuation of inclination and expansion or contraction of the slope surface. The method of measurement involves the use of such devices as invar wire extensometer (Fig. 2-7.2) and ground inclinometer or a simplified method using displacement stake or displacement plate (Fig. 2-7-3).

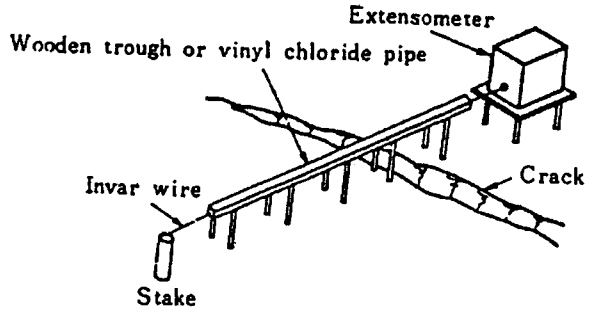


Fig. 2-7-2 Schematic Diagram of Invar Wire Extensometer

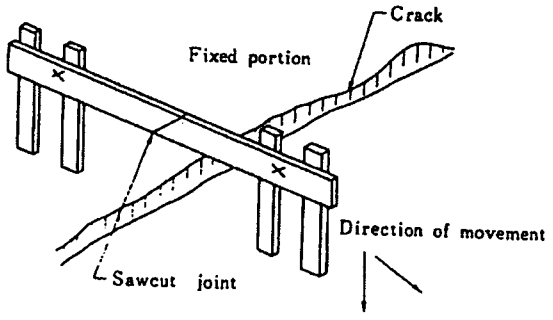


Fig. 2-7-3 Simple Displacement Plate

Sliding surfaces may be investigated through examination of the boring core or measurement of the sliding surface. Core examination is employed to verify evaluation through measurement or when the sliding surface cannot be evaluated through measurement survey. Survey through measurement of the sliding surface is undertaken by embedding rigid polyvinyl chloride pipe equipped with strain gauge (pipe strain gauge type inclinometer) or an insertion type underground inclinometer (Fig. 2-7-4) in the bore hole to measure changes in the internal stress in a ground or bore hole incline angle in order to grasp the condition of the sliding surface. The point at which the pipe strain gauge type or insertion type underground inclinometer is embedded is, in principle, all bore holes but in some cases, this may be limited to the bore holes along the main measurement line.

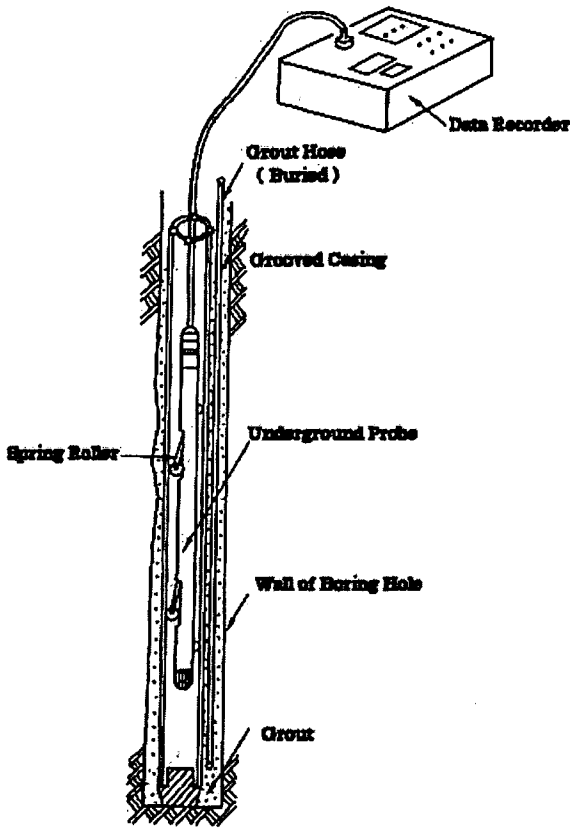


Fig. 2-7-4 Underground inclinometer

Survey of the sliding surface is undertaken when it is clear that the landslide block has been moving from the results of the site reconnaissance or measurement of ground movement.

2.8 Survey on Debris Flow

2.8.1 Checkpoints of Survey

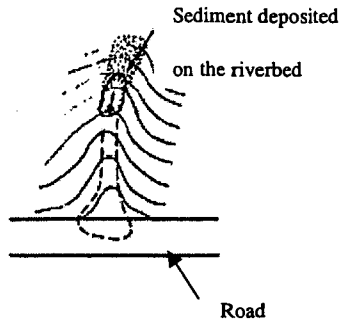
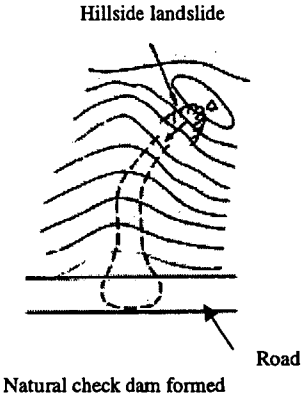
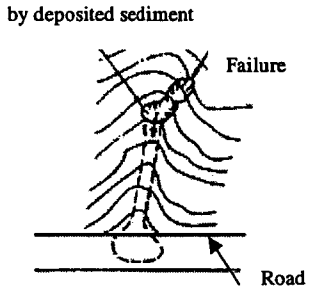
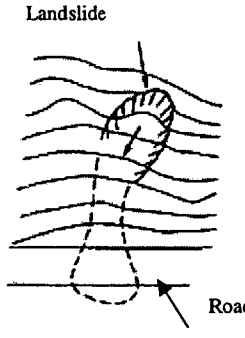
A debris flow is a phenomenon in which soil at the bottom of a valley or upstream moves as a result of heavy rain or earthquake and flows in hydraulic bores out of the valley. There are 4 main types of debris flows as shown in Table 2-8-1.

Survey on debris flow is focused on the following points and involves investigation of materials on past damage, site reconnaissance and interpretation of aerial photographs.

① Possibility of a debris flow occurrence (frequency of occurrence, weather conditions, etc.)

② Scale and characteristics of the debris flow (peak amount of discharge, the flow velocity, wave height,

Table 2-8-1 Classification of Debris Flow

Category	Description	Schematic Drawing
(1) Fluidisation of deposited sediment and gravel on the riverbed	Fluidisation of sediment and gravel deposited on the bed of a steep stream due to the supply of a large quantity of water due to a downpour or rapid thawing	
(2) Fluidisation of sediment produced by a hillside failure	Fluidisation of sediment produced by a hillside failure as the structure is broken up while sliding down the slope and mixed with water	
(3) Collapse by natural check dam	Fluidisation of sediment forming a natural check dam(s) which is created by the blockage of a mountain stream by failed sediment and which is then eroded by overflow or which collapses	
(4) Fluidisation of landslide soil mass	Fluidisation of a cohesive soil mass with a high water content originally produced by a landslide	

volume of soil, maximum gravel diameter, etc.)

③ Area likely to be inundated by the debris flow

④ Presence of existing facilities for erosion control or forestry protection and its configuration and size.

In the event a planned road would cross over a mountain stream by bridge or culvert where the occurrence of debris flow is expected, the peak amount of discharge, the flow velocity, maximum wave height and maximum gravel diameter of the debris flow should be investigated as material for reviewing whether or not the debris flow will pass without damaging the bridge or culvert.

2.8.2 Survey on Occurrence of Debris Flow

(1) Identification of Locations where Debris Flow may occur

In depressed topography to which rain water gathers, in mountain streams where water flow exists during rainfall, for mountain streams with 15 degrees or more gradient of the river bed and more than 5 ha of catchment area upstream from the point at which the gradient of the river bed is 15 degrees, with consideration focused on cases in which there is sediment at the river bed that could become debris flow, locations at which such debris flow may occur need to be investigated.

Moreover, catchment areas of less than 5ha upstream from a point at which the gradient of the river bed is 15 degrees where relatively large failure at a mountainside can be expected due to the soil properties, spring water or history of failure may be considered as areas where debris flow can be expected. The above survey should be undertaken through rough interpretation of topographic map and aerial photographs firstly and should be revised through site reconnaissance.

(2) Estimation of Frequency of Debris Flow

Existing materials on disaster records, hearings and site reconnaissance should be utilized to determine the times and frequency of occurrence of recent debris flow in the relevant mountain stream.

(3) Estimation of Rainfall Conditions Causing Debris Flow

Areas should be divided based on similarity of rainfall conditions, and materials on rainfall that caused debris flow in the past, heavy rains that did not lead to such occurrence should be collected in order to determine the distinction between rainfall conditions that lead to such debris flow and those that do not lead to such debris flow.

In the event there has been no rainfall that has caused debris flow in the relevant area, the maximum amount of rainfall at which such debris flow did not occur shall be used as the temporary rainfall conditions on the

occurrence of debris flow, and other information such as topography and soil properties should also be used in the determination.

2.8.3 Survey on Estimation of Scale, Character and Inundation Area of Debris Flow

(1) Estimation of Scale and Character of Debris Flow

In order to review the scale and positioning of countermeasures for debris flow, the scale of the debris flow (discharged soil volume), peak amount of discharge, the flow velocity and other such factors need to be estimated.

Estimation of the discharged soil volume and the maximum grain size of debris flow may be undertaken by investigating the volume of river bed sedimentation and grain size distribution through site reconnaissance, but the peak discharge, the flow velocity and unit weight of debris flow are estimated with reference given to experimental and theoretical research on debris flow and empirical formulae obtained from field observation of debris flow.

The flow velocity of a debris flow ranges from several meters to 10 meters per second in the case of flow having large gravel content (gravel type debris flow). In debris flow in volcanic areas (mud flow type debris flow), depending on water content, the velocity may range from small up to 15 to 20 meters per second. Moreover, with respect to the wave height of a debris flow, the height is generally 3 meters or less. If there are records of debris flow that have occurred close to the targeted mountain stream, such materials should be used as reference in making estimations after confirming the similarity of the topography and geology.

(2) Estimation of Inundation Area of Debris Flow

The debris flow area and its inundated area downstream from the point at which such flow may occur are estimated. Debris flows are assumed to become sedimentation covering the flat land at the bottom of valleys and alluvial fans in the present topography. While in general, the point at which sedimentation begins is assumed to be the exit of the valley, partial sedimentation may also begin where the width of the flow suddenly increases and the gradient of a river bed is 15 degrees or less, where the width of the flow becomes narrower or where the gradient of river bed suddenly becomes gentle.

In general, most mountain stream has experienced debris flow in the past. For this reason, when the exit of a valley is investigated, it is possible to know what scale of debris flow with what diameter of gravels and what extent of discharge and sedimentation have occurred in the past.

2.9 Survey on Retaining Wall and Culvert

2.9.1 Checkpoints of Survey

Structures constructed in road earth works include retaining walls, culverts.

The original ground that forms the foundation of a structure is generally complex and in particular, in recent years it has increased to construct structures on places in inferior conditions such as extremely soft ground or places with numerous undulations such as mountainous area. Moreover, in construction work in urban areas, impact on the vicinity ground or adjacent structures or the issue of vibration needs to be taken into consideration in not a few instances.

For this reason, in constructing structures, investigation needs to be undertaken on the impact to the vicinity in addition to survey of soil and geology in order to ensure design and construction of structures conforming to the conditions of the site.

The following represent the items that need to be investigated with respect to the design and construction of structures and their major checkpoints. For investigation of structures in the vicinity and construction conditions, refer to 2.11.

(1) Survey on Topography and Geology

Data on geological surveys and boring undertaken in the vicinity of the construction site should be collected, site reconnaissance should be undertaken, and the dip of the geological stratum and surface, difference in elevation, undulation, the existence of spring water, system, direction and volume of runoff water need to be reviewed.

(2) Investigation on Soil Properties

The composition of the strata, soil properties and conditions of the ground water at the construction site need to be grasped, and in order to obtain materials required for determining the type of foundation to be used for

the structure, soil tests need to be performed with respect to the following factors depending on the type of structure.

① Investigation for soil parameters required for calculating external force (earth pressure) (unconfined compression and triaxial compression tests, unit weight test, etc.)
② Investigation for soil parameters required for stability analysis (unconfined compression and triaxial compression tests, unit weight test, standard penetration test, etc.)
③ Investigation for soil parameters required for calculating the bearing capacity of shallow foundation and design of pile foundation (triaxial compression test, standard penetration test, plate loading test, etc.)
④ Investigation for soil parameters required for calculating consolidation settlement (natural water content test, consolidation test, etc.)
⑤ Investigation concerning the ground water (boring survey, in-situ permeability test, etc.)

The above investigation items should be adapted to general structures and in the event that the structure is of a large scale or involves special construction, or the ground is extremely soft, other tests should be added as required to obtain soil parameters needed for the design work.

2.9.2 Survey on Retaining Wall

The structural design of a retaining wall differs by the topography, soil, geological properties, impact on structures in the vicinity, construction conditions and height of the retaining wall. Moreover, with respect to retaining walls in mountainous areas, retaining walls built on soft ground or retaining walls built in confined space, there are cases in which special structural design needs to be adopted. For this reason, concerning the survey on retaining walls, it is important to give consideration to these points.

Moreover, in the event a retaining wall is constructed on soft ground or inclined ground, it is necessary to care lateral movement through embankment loading or displacement or overturning of the retaining wall. Furthermore, it is necessary to care floating up of U shaped retaining walls used in open-cut roads. In such cases, adequate investigation on soil properties need to be undertaken with respect to the soil properties, structure of the strata, inclination of bedrock, condition of ground water and of the backfill soil and the stability of the retaining wall needs to be studied based on such results.

2.9.3 Survey on Culvert

In the design and construction of a culvert, it is necessary to take into consideration not only aspects as a culvert

itself but also the impact on the flatness of the road surface and the behavior of embankments both sides of the culvert.

In the event of common ground, there is a very rare case of the culvert being damaged or having a large differential settlement at the adjoining point to the embankment as a result of the settlement and deformation of the ground. However, in the event of soft ground or in locations where the soil properties differ significantly, differential settlement due to the load of the embankment may occur and which causes a difference in road level or cracks in the culvert or damage of the culvert. For this reason, with this type of ground, it is necessary to investigate the soil properties and the strata composition in order to design the culvert foundation.

In the event the material is inappropriate or the construction work is poor with respect to the foundation or backfill of a culvert, the culvert may settle locally or the backfill in the vicinity of the culvert may settle thus negatively affecting the flatness of the road surface. For this reason, the quality of the backfill materials used needs to be evaluated through tests for categorizing the soil or through compacting test.

2.10 Soil Investigation

2.10.1 Soil Investigation, Test Methods and Relevant Standards

The soil investigation and test methods and the relevant standards which are commonly used for investigations for road earthwork at present are listed in Appendix.

As new technologies, methods and equipment are constantly being developed for the soil investigation and test, the selection of the most appropriate methods is necessary taking the objectives, required information and accuracy, reliability and cost of the investigation and test methods into consideration.

2.10.2 Interpretation and Utilisation of Results

(1) Geophysical Exploration

(i) Elastic Wave Exploration

Elastic wave exploration estimates the soil properties based on the velocity of the seismic wave travelling through the ground. Vibration pick-ups are installed on lines to observe the travelling time of the vibrations caused by a dynamite explosion to vibration each pick-up and the underground conditions are analysed based on the obtained records.

In general, the harder a substance is, the faster the velocity of the elastic wave of that substance is. Utilising this property, elastic wave exploration is used to analyse the state of stratification of soil, soft rocks and hard rocks as basic data to examine the workability of a cutting section, to examine the stability of a cut slope and to judge the suitability of soil as a banking material. The general velocity of the P wave of the ground is shown in Fig. 2-10-1.

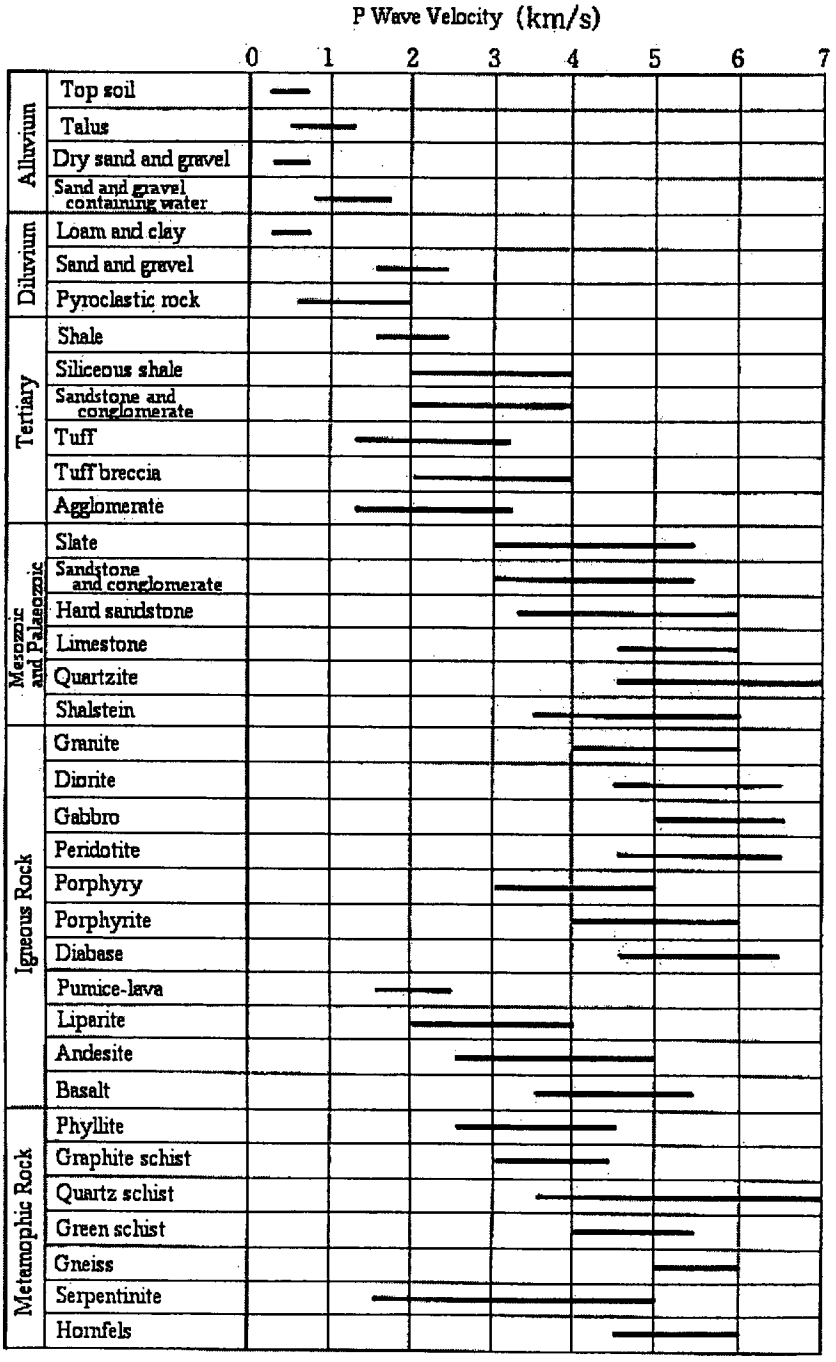


Fig. 2-10-1 In-Situ Elastic Wave Exploration of Major Formations and Rocks

(ii) Electric Prospecting

The resistivity method judges the geological conditions based on the recorded electric resistivity to an artificial current sent to the ground. Although the resistivity of the ground is affected by many factors, including the particle size, porosity and water content of the soil and rocks forming the ground, it is particularly affected by the water content. As a general tendency, certain degrees of correlation exist between the geology and resistivity as described below.

① In the case of sedimentary layers, the resistivity increases from a clayey layer to a sandy layer and a gravelly layer in that order. The actual value considerably varies from several Ω–m for a certain clayey layer to several ten thousands of Ω–m for fresh, hard and close bedrock.

② The resistivity is generally higher for dry soil than wet soil. In regard to water itself, the resistivity increased from confined (deep) groundwater to free (shallow) groundwater and surface water in that order.

③ The resistivity of permeable layers in the aquiferous state tends to be in the range of 100 – 1,000 Ω–m except for gravelly layers in an alluvial fan.

(2) Sounding

(i) Standard Penetration Test

The relative density of sand and degree of consistency of clay can be classified based on the N values shown in Table 2-10-1. Meanwhile, the relationship between the N value and the cohesion of clayey soil, the internal friction angle of sandy soil and the bearing capacity of the ground are shown in Figs. 2-10-2 and 2-10-3.

Table2-10-1 Relation between N Value and Relative Density of Sand, Consistency of Clay

Sand			Clay		
N Value	Relative Density	Dr	N Value	Consistency	q (kgf/cm^3)
0 – 4	Very loose	0.0 – 0.2	< 2	Very soft	< 0.25
4 – 10	Loose	0.2 – 0.4	2 – 4	Soft	0.25 – 0.5
10 – 30	Medium	0.4 – 0.6	4 – 8	Medium	0.5 – 1.0
30 – 50	Dense	0.6 – 0.8	8 – 15	Hard	1.0 – 2.0
> 50	Very dense	0.8 – 1.0	15 – 30	Very hard	2.0 – 4.0
			> 30	Solid	> 4.0

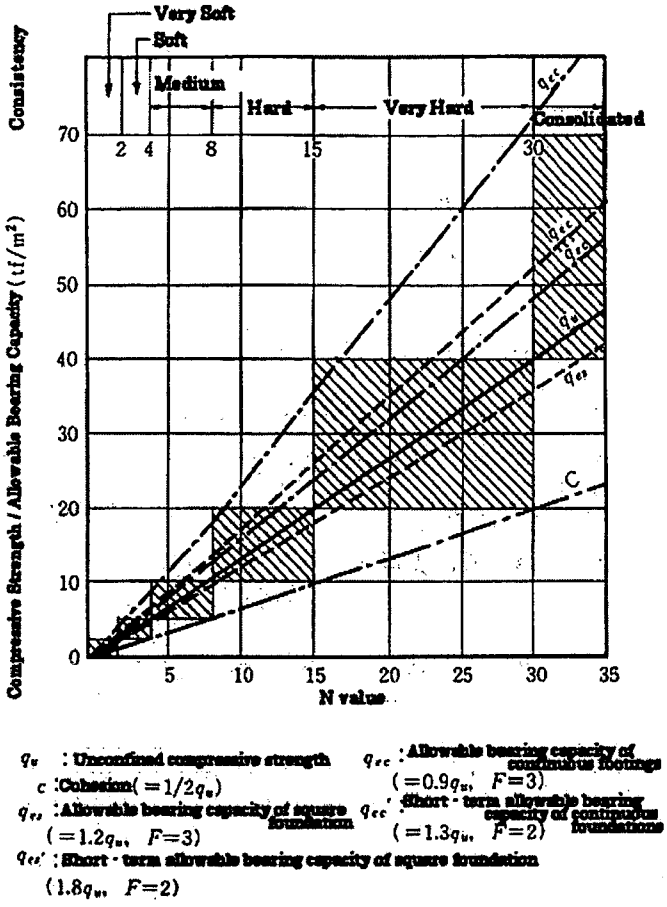


Fig. 2-10-2 Relationship between N value and Consistency / Unconfined Compressive Strength and Allowable Bearing Capacity of clay

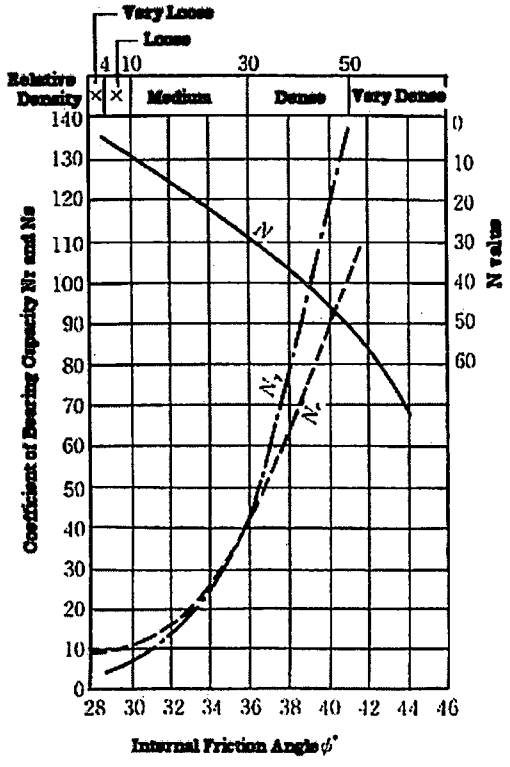


Fig. 2-10-3 Relationship between N value and Internal Friction Angle ϕ, Coefficient of Bearing Capacity

In the standard penetration test, the ratio between the potential energy of the hammer and the energy conveyed to the rod immediately after impact is called the percussion efficiency. This percussion efficiency varies depending on the test apparatus or test method and considerably affects the N value. Proper attention must be paid to the percussion efficiency, therefore, when using the test results.

(ii) PWRI (Public Works Research Institute) Type Cone Penetration Test

This test is mainly used for surveys on weathered layers at slope failure sites because (a) the relatively light weight test apparatus allows a large number of penetration tests on a slope in a short period of time and (b) the small impact energy enables the detection of minute changes of the penetration resistance of the stratum. Based on the number of percussions required to penetrate by 10 cm (N_d), three layers can be identified: surface soil layer ($N_d < 5$), weathered layer ($5 < N_d < 30$) and bedrock ($N_d > 30$).

(iii) Swedish Weight Sounding Test

In this Swedish weight sounding test, the resistance value as a test result is expressed by the weight load required for static penetration (W_{sw}) and the number of semi-rotations converted per 1m of penetration when penetration is achieved by rotating the screw after the end of the static penetration process (N_{sw}). A stratum of which the resistance can be expressed by W_{sw} consists of very soft or soft clayey soil or very loose sandy soil in most cases. The relations between N_{sw} and the N value of the standard penetration test and between W_{sw} & N_{sw} and the unconfined compressive strength of clayey soil are shown in Figs. 2-10-4 and 2-10-5 respectively.

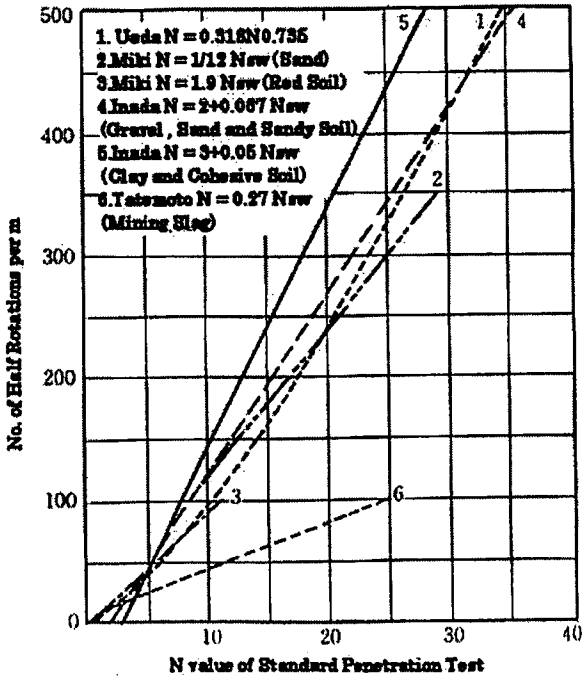


Fig. 2-10-4 Relationship between N_{sw} value and N value

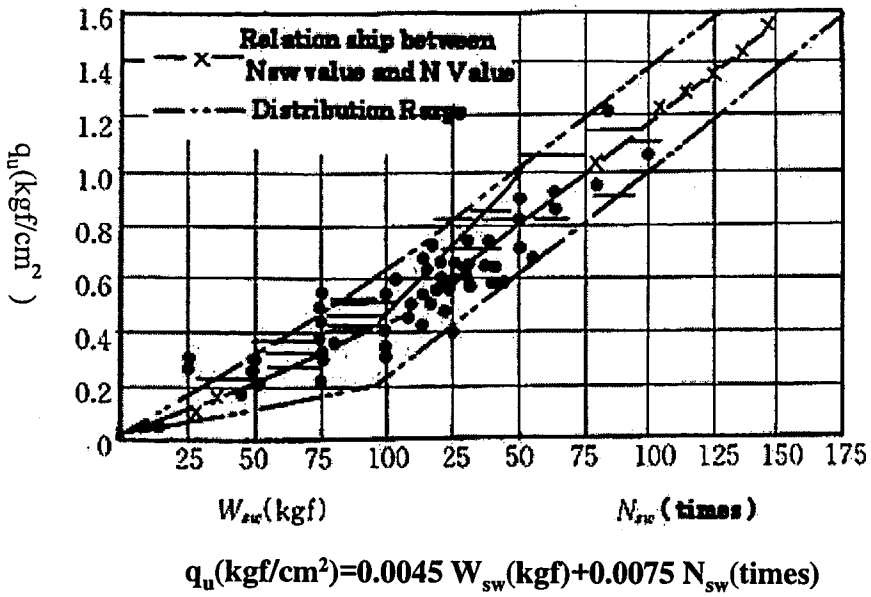


Fig. 2-10-5 Relationship between W_{sw}, N_{sw} and q_u

(iv) Dutch Cone Penetration Test

The relation between the cone penetration resistance q_c and the N value for sand and gravel and the relation between q_c and the N value for clay are shown in Figs. 2-10-6 and 2-10-7 respectively.

q_c generally has the following relation with the unconfined compressive strength q_u.

$$q_c = (10 \sim 15) q_u = (20 \sim 30) c_u \qquad \text{..... (2.10.1)}$$

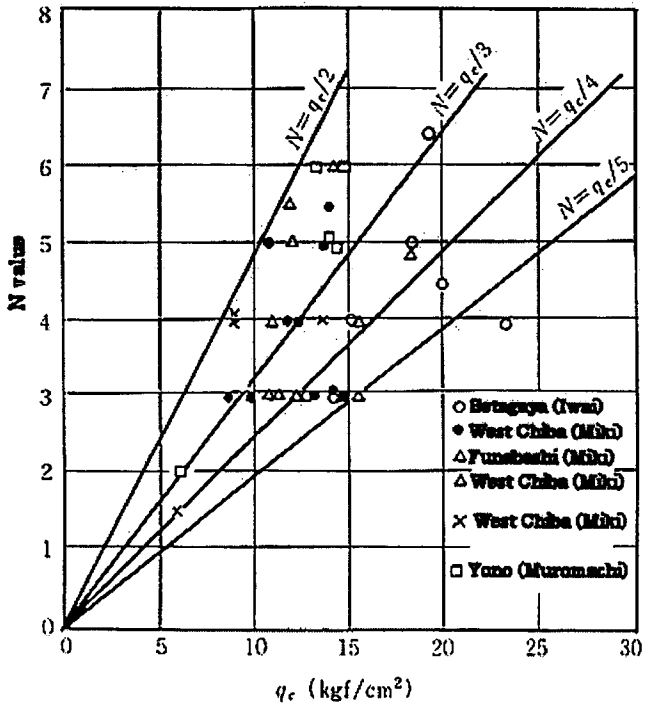


Fig. 2-10-6 Relationship between q_c value and N value for Sand and Gravel

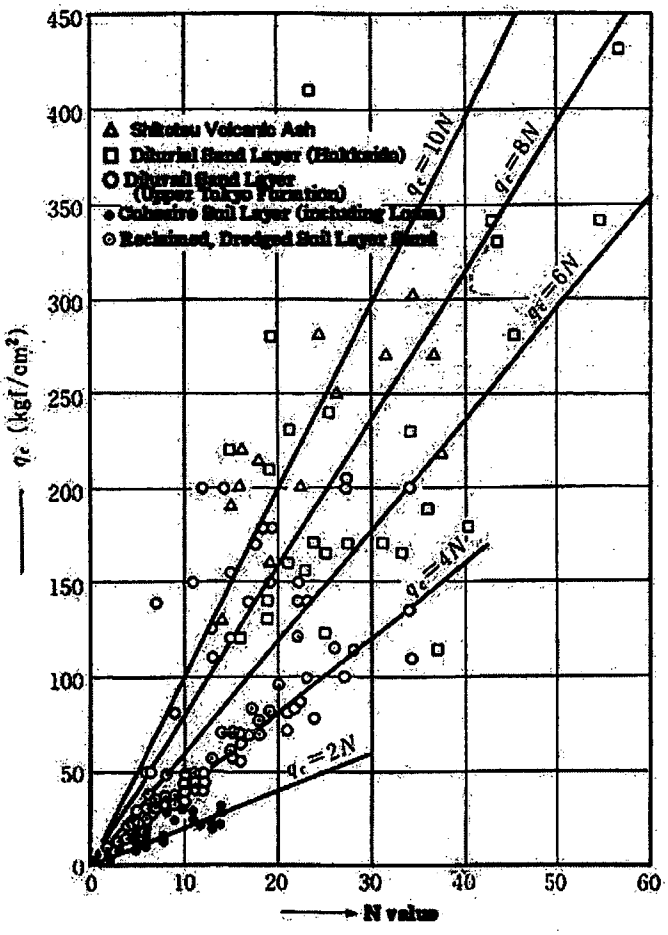


Fig. 2-10-7 Relationship between q_c value and N value for Clay

(v) Portable Cone Penetration Test

In the portable cone penetration test for clayey soil, the cone penetration resistance q_c and the unconfined compressive strength q_u generally show the following relation.

$$q_c \approx 5q_u = 10c_u \qquad \text{..... (2.10.2)}$$

(3) Loading Test

(i) Borehole Loading Test

The modulus of deformation E obtained by the borehole horizontal loading test and the N value obtained by the standard penetration test has the following relation in an approximate manner regardless of the ground materials.

$$E = 700N \quad kN/m^2 \qquad \text{..... (2.10.3)}$$

The modulus of deformation obtained by the borehole horizontal loading test is roughly identical to the modulus of deformation obtained by the triaxial compression test and the unconfined compression test regardless of the ground materials.

Meanwhile, the coefficient of horizontal subgrade reaction k_H used for foundation design can be calculated using the modulus of deformation obtained by the borehole horizontal loading test.

(ii) Plate Loading Test for Ground

The plate loading test results are used to examine settlement and bearing capacity of the bearing stratum for the foundation based on the relation between the loading pressure of the plate and settlement. However, settlement and the bearing capacity of the foundation is affected by many factors, including the depth of embedment, shape, size and rigidity of the foundation, ground composition, groundwater level, surcharge and duration of loading. Fig. 2-10-8 is 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.

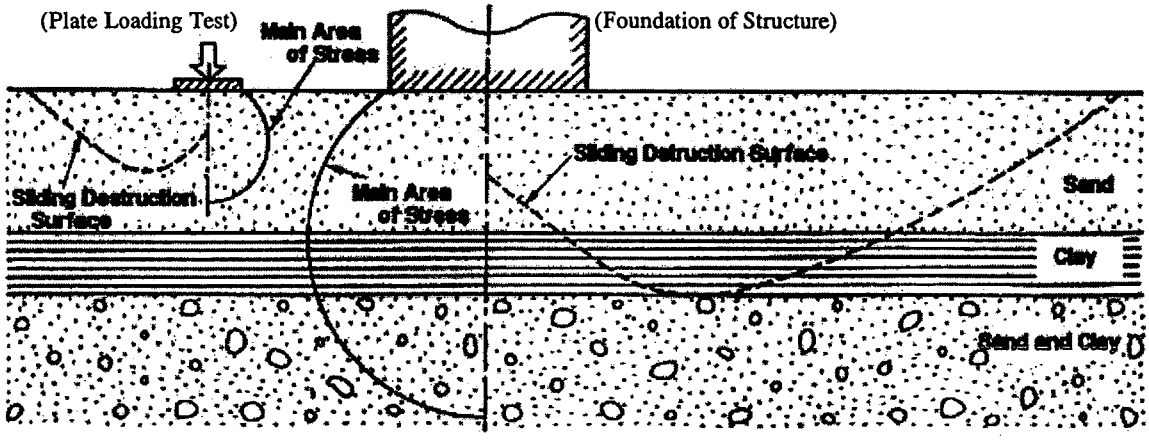


Fig. 2-10-8 Relationship of Size between Foundations for Structnre and Loading Plate

(iii) Plate Loading Test for Road

When the base course thickness is calculated using the bearing capacity coefficient K_{30} (coefficient of subgrade reaction for a plate of 30 cm in diameter) for the design of the cement concrete pavement, the required thickness of material to secure the necessary bearing capacity coefficient of the base course K_1 is calculated in correspondence with the bearing capacity coefficient of the subgrade K_2 using the design curve shown in Fig. 2-10-9.

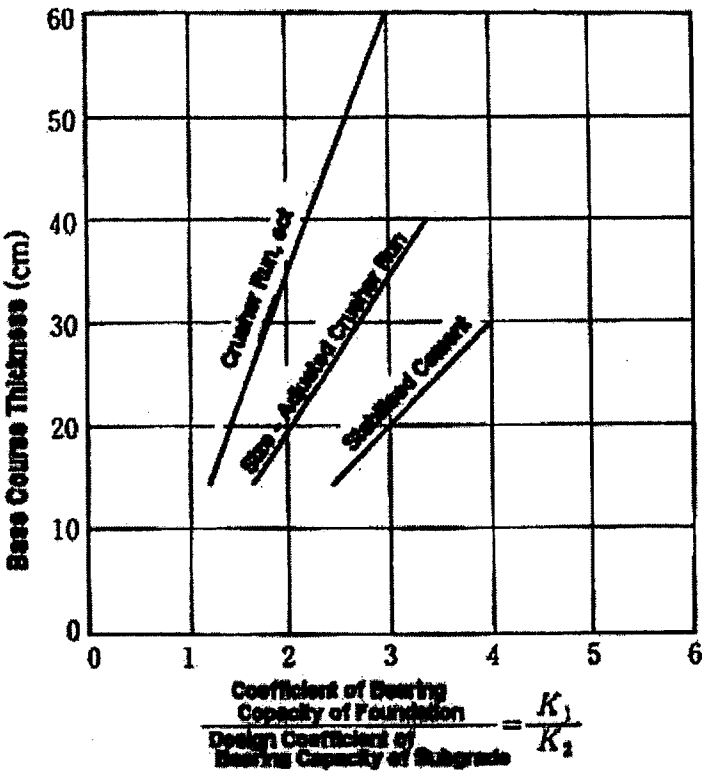


Fig. 2-10-9 Base Course Design Curve

(iv) In-Situ CBR Test

The in-situ CRB test is used as execution management test to confirm the CBR used for the evaluation and selection of the base course materials.

(4) In-situ Density Test

The generally used standard values for compaction as indices for the quality control of an embankment are the degree of compaction in the case of an embankment made of coarse grained soil and the air void ratio or degree of saturation in the case of an embankment made of fine grained soil.

The degree of compaction is generally indicated by the expression below and the dry density of an embankment is obtained by the in-situ density test. The maximum dry density for the laboratory compaction test can be obtained based on JIS (Japanese Industrial Standards) A1210.

$$\textit{Degree of Compaction} = \textit{Dry Density of Embankment} \,/\, \textit{Maximum Dry Density in Laboratory Compaction Test} \times 100\,(\%)$$

The air void ratio v_a and the degree of saturation S_r can be indicated by the following expression where the unit weight of the soil particles must be known in addition to the dry density and the water content, both of which are obtained by the in-situ density test.

$$v_a = 100 - \frac{\rho_d}{\rho_w} \times \left(\frac{100}{\rho_s} + w \right) \quad (\%) \qquad \text{..... (2.10.4)}$$

$$S_r = w \Big/ \left(\frac{\rho_w}{\rho_d} - \frac{1}{\rho_s} \right) \quad (\%) \qquad \text{..... (2.10.5)}$$

Where,

ρ_d : dry density of the embankment (g/cm^3)

ρ_w : unit weight of water (g/cm^3)

ρ_s : unit weight of soil particles (g/cm^3)

w : water content (%)

(5) Field Measurement

The ways at soft ground and landslide sites are explained in 4.4.2 and 3.8.1 respectively and a measurement example using pipe strain gauge type inclinometers is shown in Fig. 2-10-10.

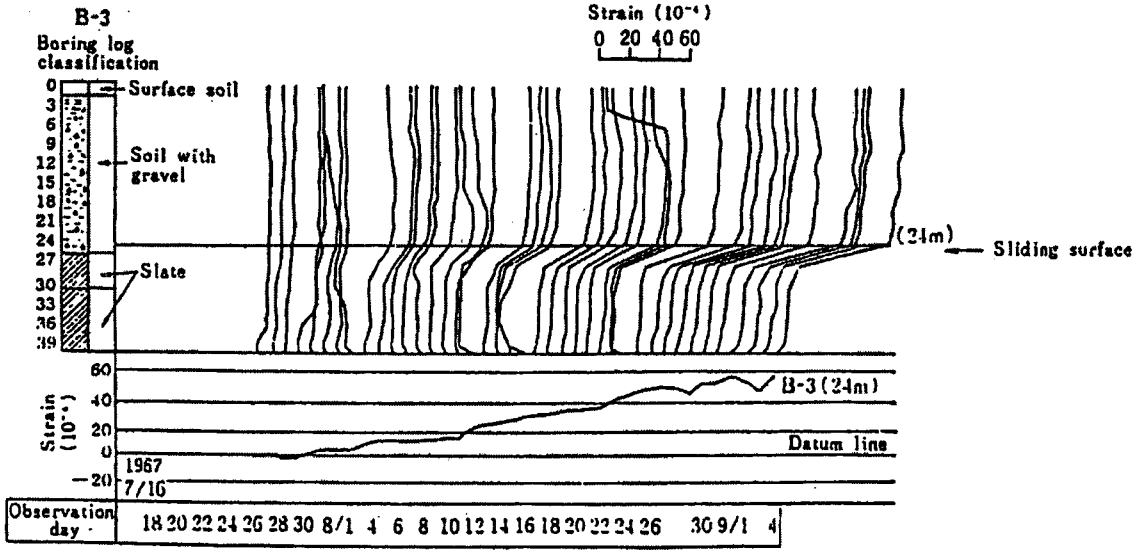


Fig. 2-10-10 Example of Cumulative Diagram of Strain Fluctuations Measured by Straingauge Type Inclinometer

(6) Soil Tests

(i) Specific Gravity Test of Soil Particle

The specific gravity test of soil particle is used to calculate the air void ratio and the degree of saturation in the compaction test (see Expressions 2.10.4 and 2.10.5). It is also used to calculate the basic soil properties, such as the void ratio e and the dry density ρ_d.

$$e = \frac{\rho_w}{\rho_d}\rho_s - 1 \qquad \text{..... (2.10.6)}$$

$$\rho_d = \rho_w / \left(1/\rho_s + w/S_r\right) \qquad \text{..... (2.10.7)}$$

The unit weight of the soil particle generally falls between 2.5 and 2.8 and the selection of ordinary soil with a unit weight of soil particle of around 2.65 does not pose any major problems. Soil containing organic matters tends to show 2.5 or lower while peat generally shows around 2.3 or lower. Volcanic cohesive soil mainly falls between 2.7 and 2.8 but some soil is observed to have around 2.9.

(ii) Water Content Test

The water content is an important indices of the soil properties and the natural water content teaches us the types of soil to a certain extent. For example, soil with a natural water content of not more than 20 – 30% can be considered to be sandy soil. Similarly, soil with a natural water content of more than 40 – 50% and more than 200% can be considered to be clayey soil and organic soil respectively. Such volcanic cohesive soil as Kanto loam generally shows a natural water content of some 80 – 150%.

(iii) Grain Size Analysis

The particle size of soil is used for soil classification. In the case of coarse soil and sandy soil, it is used as an index to show the engineering properties, such as permeability, compatibility and bearing capacity.

The state of the particle size distribution of soil can be established by calculating the uniformity coefficient U_c and the coefficient of curvature U_c' shown in the following expressions on the basis of particle sizes D_{60}, D_{30} and D_{10} (effective size) corresponding to the passing percent weight of 60%, 30% and 10% respectively of the grain size distribution curve which are obtained by the grain size analysis.

Uniformity coefficient $\quad U_c = D_{60}/D_{10} \qquad \text{..... (2.10.8)}$

Coefficient of curvature $\quad U_c' = \left(D_{30}\right)^2 / \left(D_{10} \times D_{60}\right) \qquad \text{..... (2.10.9)}$

A higher uniformity coefficient means a wider particle size distribution. In general, soil of which the U_c is less than 4 – 5 indicates "a poor particle size distribution" while soil of which the U_c is 10 or higher indicates "a good particle size distribution".

The coefficient of curvature is used to quantitatively show the stepped particle size distribution and a value of U_c' from 1 to 3 indicates "a good particle size distribution".

It is known that the relation shown by Expression (2.10.10) roughly exists between the coefficient of permeability k and the effective size D_e (meaning particle size D_{10} corresponding to a passing percent weight of 10% in the grain size distribution curve) (Hazan's expression).

Coefficient of permeability $k\ (cm/\sec) = C \cdot D_e^{\ 2}$ ..... (2.10.10)

Where,

C : constant (100 – 150) (1/cm · sec)

D_e : effective size (cm)

This expression is said to be applicable to sand of which the particle size is relatively uniform.

(iv) Liquid Limit and Plastic Limit Tests

It is known that the engineering properties of fine grained soil, including clay, significantly change depending on the soil structure and consistency. Fine grained soil, excepting highly organic soil, is classified based on its location on the plasticity chart (see Fig. 2-10-14), in turn based on the consistency characteristics.

When the natural water content w_n of soil is larger than the liquid limit w_L, soil becomes muddy, making the careful selection of construction machinery and construction method necessary.

The consistency index ($I_C = (w_L - w_n)/(w_L - w_p)$) means the relative hardness or stability of cohesive soil. In the case of $I_C \geq 1$, the natural water content is near the plastic limit w_p or lower, indicating relatively stable state. When $I_C = 0$, the natural water content is near the liquid limit, meaning that disturbance of the soil could lead to liquefaction.

(v) Test on Chemical Properties

Problematic soil from the chemical point of view is generally strongly acid or alkaline soil, highly organic soil and chloride-rich soil. This soil has the following impacts on road earthwork.

① Strongly acid soil: Special attention should be paid to soil of which the pH value is 4 or lower as it is unsuitable for slope revegetation. Soil containing volcanic sulphides could corrode such steel materials as steel piles and corrugated pipes.

② Strongly alkaline soil: Special attention should be paid to soil of which the pH value is 8.5 or higher as it is unsuitable for slope revegetation. This type of soil could destroy the paint coat.

③ Highly organic soil: It is said that when the organic content is higher than 2 – 4%, the specific gravity, consistency and compressive/shear strength are affected.

④ Chloride-rich soil: This soil facilitates the corrosion of steel materials buried in the ground.

(vi) Compaction Test

The compaction test result is used as the reference value to determine the degree of compaction as well as the water content during the construction of an embankment. Refer to (4) – In-Situ Density Test for the test method.

(vii) CBR Test

This test is conducted to determine the bearing capacity index of the subgrade soil or base course material (mainly granular base course material) and is widely used to decide the thickness of various types of pavement and also to judge the suitability of soil as a base course material for different types of pavement.

The design CBR for the subgrade is used to decide the pavement thickness while the modified CBR for the base course material is used to judge the suitability of soil as a base course material for a different type of pavement.

(viii) Unconfined Compression Test

The unconfined compression test is mainly used (a) to estimate the undrained shear strength of a soil sample which is collected from natural ground and which is not disturbed at its original position based on the unconfined compressive strength of the said sample and (b) to judge the improvement effects and to evaluate the stability of the improved ground by clarifying the compressive strength of soil which is artificially improved by means of in-situ compaction or chemical treatment. This test can be conducted with self-supported soil samples. As the testing apparatus is easy to operate, this test is suitable for the testing of many samples.

Using the unconfined compressive strength q_u, the undrained cohesion of clayey soil c_u can be determined by Expression (2.10.11).

$$c_u = q_u/2 \qquad \text{..... (2.10.11)}$$

(ix) Triaxial Compression Test

c_u and ϕ_u determined by the unconsolidated-undrained (UU) test are used to examine the stability of saturated cohesive soil ground and rapid banking work. In contrast, c' and ϕ' or c_{cu} and ϕ_{cu} determined by the consolidated-undrained (CU) test are used to examine the stability of embankments and cut slopes and also to examine the stability of structures liable to the pore water pressure of spring water and seepage water.

The rate of strength increase m is used to examine the stability of embankments and earth structures constructed on soft ground during or after construction work. This m is calculated by Expression (2.10.12) using the ϕ_{cu} obtained by the CU test where the consolidated stress p is altered.

$$m = c/p = \sin\phi_{cu} / (1 - \sin\phi_{cu}) \qquad \text{..... (2.10.11)}$$

Where,

c : cohesion, taking the strength increase by consolidation into consideration

c_d and ϕ_d obtained by the consolidated-drained (CD) test are used to examine the stability of cut slopes made up of sandy soil or sand with a high permeability.

(x) Consolidation Test

The results of the consolidation test are used to estimate the settlement and the consolidation velocity of saturated fine grained soil by loading. They are also used to examine the consolidation acceleration effects of the counter-measures against soft ground. The calculation and examination methods of this test are described in 3.6.

(xi) Laboratory Permeability Test

The permeability test is conducted to calculate the uplift acting on structures below the groundwater level, to select an appropriate drainage method for excavation and other types of work, to select a cofferdam construction method and filling materials, to design subsurface drainage facilities and to calculate the coefficient of the permeability of soil in relation to judgement on the suitability of materials used for sand drains and sand mats. However, as estimation of the coefficient of permeability for natural ground is problematic based on the results of this test, it is appropriate to consider this test as being unsuitable to measure the coefficient of the permeability of the ground. Laboratory test results for gravelly soil in particular should be treated as simple yardsticks.

2.10.3 Classification of Rocks and Soil

The classification of rocks and soil is necessary to establish rough ideas about rock and soil properties for the purpose of design and construction work. Standardisation of the names of rocks and soil used in these Guidelines allows accurate understanding of its contents on common ground.

Cases where the classification names of rocks and soil are actually required are listed below.

① Indication of soil investigation results (preparation of soil and geological profile and soil property chart)
② Judgement on standard slope gradients for cut slopes and embankments
③ Judgement on material suitability (judgement on spoil and selection of subgrade soil and backfill soil)
④ Judgement on bulking factor of soil
⑤ Selection of construction method and construction machinery (selection of excavation method and planning for excavation)
⑥ Calculation of working capacity of construction machinery
⑦ Planning of slope protection and retaining walls, etc.

(1) Classification of Rocks

(i) Investigation for Classification of Rocks

Many methods are used to classify rocks and there is no unified classification method. In the case of such road earthwork as slope excavation, the difficulty of excavation is generally used as criteria for rock classification.

The excavation difficulty of the bedrock is determined by the degree of such breaking loads as the compressive, tensile and shearing loads applied by machinery to the bedrock and the strength of the bedrock to resist these forces. Accordingly, it is inferred that the workability of excavation work is mainly determined by the strength of the bedrock, i.e. hardness of rocks, and the state of fissures, such as joints, in the bedrock. In general, the following methods are used to investigate the hardness of rocks and the state of fissures in the bedrock.

① Investigation of work records
② Field survey
③ Boring
④ Elastic wave exploration
⑤ Compression test on samples
⑥ Measuring of elastic wave velocity with samples

⑦ Degree of repulsion of Schmidt hammer

(ii) Rock Classification Criteria

When classifying rocks for the purpose of excavation, rocks mainly requiring blasting tend to be called hard rocks. Meanwhile, rocks mainly excavated by a ripper are called soft rocks and rocks which permit work by a bulldozer are called soil. Table 2-10-2 shows the classification of rocks and soil based on the excavation difficulty. In principle, the classification of rocks should be based on the state of rock quality and classification based on elastic wave exploration should be used for reference purposes. The classification results are used to determine a suitable excavation method as described earlier and are also used to determine the cut slope gradient for the bedrock.

Table 2-10-2 Classification of Soil and Rock Based on Ease or Difficulty of Excavation

Name	Description	Remarks	Correspondence to Simplified Soil Classification in Japan Unified Soil Classification
[Rock or Stone]			
Hard rock	None or hardly any cracks with good adherence	Elastic wave velocity: 3,000 m/sec or higher	
Medium hard rock	Less advanced weathering (crack interval of some 30 – 50 cm)	Elastic wave velocity: 2,000 – 4,000 m/sec	
Soft rock	Well consolidated quaternary formation; Tertiary or earlier formations with advanced weathering; that which can be excavated by a ripper	Elastic wave velocity: 700 – 2,800 m/sec	
Boulders	The concentration of boulders of various sizes makes excavation very difficult		
Rock mass; cobblestones	The inclusion of rock mass and cobblestones makes excavation difficult and tends to produce voids in a bucket	Soil with cobblestones; rock mass; crashed rock, stony riverbed	
[Soil]			
Soil with gravel	The inclusion of gravel reduces the excavation efficiency	Sand with much gravel; sandy soil with much gravel; cohesive soil with much gravel	Gravel (G) Gravelly soil (GF)
Sand	Material which is unlikely to make a heap in a bucket, etc.	Sand of coastal dunes; decomposed granite soil	Sand (S)
Ordinary soil	Easy to excavate and to create a heap in a bucket with few voids	Sandy soil; decomposed granite soil; loam with excellent particle size distribution	Sand (S) Sandy soil (SF) Silt (M)
Clayey soil	Likely to stick to a bucket, etc., producing few voids; likely to create a trafficability problem	Loam Clayey soil	Silt (M) Clayey soil (C)
Clayey soil with high water content	Likely to stick to a bucket, etc. and very poor trafficability	Loam with poor conditions; cohesive soil with poor conditions; volcanic cohesive soil	Silt (M) Clayey soil (C) Volcanic cohesive soil (V) Organic soil (O)
Organic soil			Highly organic soil (Pt)

Note: While the above description applies to those types frequently observed, special attention must be paid to the fact that the types of soil can significantly change depending on the conditions.

(2) Classification of Soil

The classification of soil follows the JGS (Standard of Japanese Geotechnical Society) 0051 "Method of Classification of Geomaterials for Engineering Purposes" of The Japanese Geotechnical Society. Geomaterials are classified by particle size using the particle size boundaries shown in Fig. 2-10-11 and the particles in each particle size range are called by the names shown in Fig. 2-10-11.

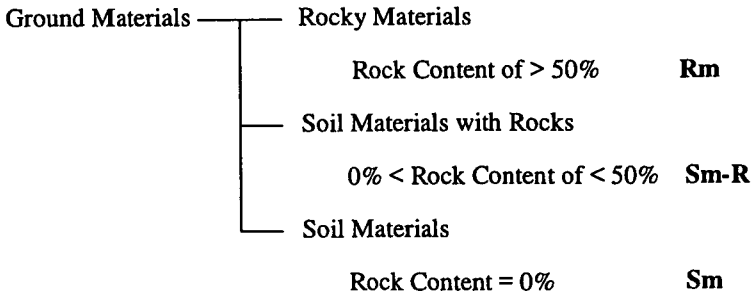
Particle Size (mm)

0.005	0.075	0.25	0.85	2	4.75	19	75	300
Clay	Silt	Fine Sand	Medium-Size Sand	Coarse Sand	Fine Gravel	Medium-Size Gravel	Coarse Gravel	Cobbles	Boulders
		Sand			Gravel			Stones	
Fine Grain Content		Coarse Grain Content						Stone Content	

Fig. 2-10-11 Particle Size Categories of Geomaterials and Their Names

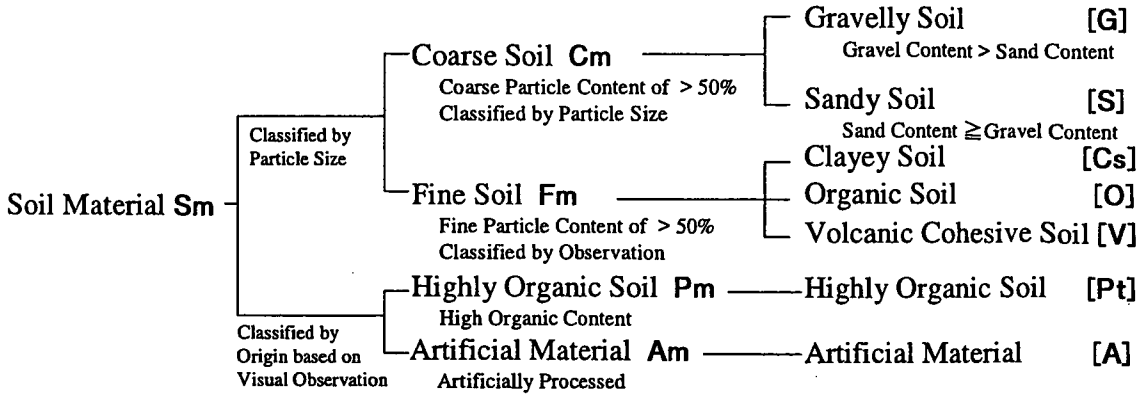
Geomaterials can also be classified based on the grain size distribution which is mainly determined by visual observation and grain size analysis using Figs. 2-10-12 and 2-10-13. However, the secondary classification of coarse grained soil is mainly based on the grain size distribution of the soil materials while the secondary classification of fine grained soil is mainly based on the plasticity chart (Fig. 2-10-14).

The coarse grain fraction in the figure means the proportion of particles (grains) of 75 μm to 75 mm in size while the fine grain fraction means the proportion of particles (grains) of less than 75 μm in size. The gravel fraction means the proportion of gravels of 2 mm to 75 mm in size and the sand fraction means the proportion of sand of 75 μm to 2 mm in size.



Note: The content (%) means the mass percentage in the ground material.

Fig. 2-10-12 Engineering Classification System for Geomaterials (Primary Categories)



Note: The content (%) means the mass percentage in the soil material

Fig. 2-10-13 Engineering Classification System for Soil Materials

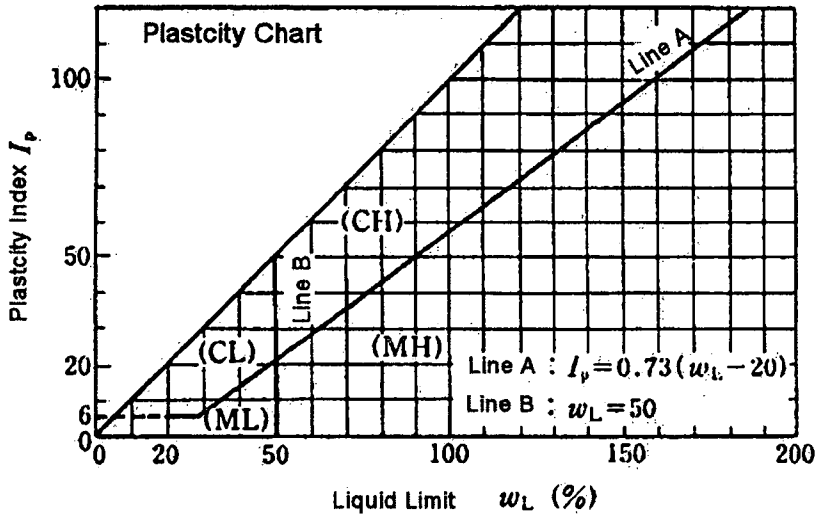


Fig. 2-10-14 Plasticity Chart

2.11 Survey on Other Issues

2.11.1 Survey on Weather Condition

With respect to the weather condition, there are surveys that are necessary for reviewing the structure of the road and construction method and determining the construction schedule at the construction stage, and that are necessary for maintenance and disaster prevention.

(1) Temperature

Temperature is an important survey factor for determining the appropriate construction system such as the construction method, quality management, and construction schedule. In general, the monthly average

temperature, the annual range of the temperature between the hottest month and coldest month and temperature type that differs by region are compiled and used as fundamental material in planning and design.

(2) Rainfall

The materials on rainfall required in road earthwork are intensity of rainfall and number of days of rain. The intensity of rainfall is expressed in terms of hourly rainfall depth and continuous rainfall depth. Hourly rainfall depth is utilized as material for the design of drainage facilities and continuous rainfall depth is utilized as material for the stability analysis of the body of road and slope. The volume of rainfall changes by region, topographical conditions and elevation and these factors need to be surveyed in advance. Rainfall during the construction stage exerts a large impact on the construction schedule, method of construction and quality management. In particular, with respect to rainfall during the construction stage, a temporary drainage system, countermeasure for failure of cut slope and embankment and muddy water that flow out from the site need to be reviewed in order to prevent reworking or negative impact on third parties.

(3) Wind

Survey on wind involves surveying the primary wind direction and wind intensity throughout the year and reviewing how the road to be constructed will impact on the vicinity. There are possibility that the surface soil dries during construction may cause dust damage to the vicinity of the site and for this reason, consideration needs to be given to sprinkling water or the use of blown sand prevention net.

(4) Waves

With respect to waves, the wave height, scouring and movement of the shoreline are the primary factors to be surveyed. For this reason, understanding of the relative position of the sea and the road is important in undertaking a survey, and in order to gain this understanding, it is necessary to survey the shoreline distance, the gradients of seashore and sea bed. Other surveys involve collection of existing materials on Typhoons and tidal waves and reviewing scouring, breaking waves and vanishing waves.

2.11.2 Survey on Environment

In the construction of a road, the impact on the environment should be forecasted from the planning stage and adequate review of measures should be undertaken for proceeding the smooth implementation of the construction work. To this end, a detailed survey that is in conformance with the regional characteristics is required.

(1) Survey on the Natural Environment

Surveys on the topography, geology, plants and animals constitute surveys on the natural environment. With respect to the topography and geology, since changes occur as a result of cutting or filling of the original ground, a survey of the impact of such work needs to be undertaken and in particular, a survey of the impact of cuts on ground water is important. With respect to plants and animals, a survey needs to be conducted on the impact of changes in the topography or changes in the ground water on such plants and animals and a review on the road structure and construction method towards environmental preservation needs to be undertaken.

(2) Survey on the Natural Scenery

In the event changes in the topography are large, review to ensure that the structure is in conformity with the region needs to be made.

(3) Survey on the Social Environment

Noise, vibration, water contamination as a result of the discharge of mud, scattering of soil and dust as a result of the transport of soil, ground settlement as a result of embankment, drying up of water as a result of cutting and other impact negatively affect the social environment around the construction site and this can be an impediment to the implementation of the construction work. In planning and construction, detailed care needs to be taken with respect to the construction method, selection of construction machinery in order to protect the social environment and ensure smooth implementation while giving consideration to related laws.

2.11.3 Survey on Cultural Assets

With respect to buried cultural assets, areas containing such cultural assets should be avoided to the extent possible from the planning stage of the road. In the event routing the road through such areas is unavoidable, adequate consultations need to be undertaken with concerned agencies and surveys conducted. In regions where there are numerous areas with buried cultural assets, construction needs to ensure care with respect to the strata and uncovered assets. When buried cultural assets are discovered during construction work, the construction should be immediately suspended and necessary measures undertaken.

2.11.4 Other Surveys

(1) Survey on Housing

Embankment on soft ground or construction of underground facilities or tunnels may cause settlement of the ground. In such cases, since housing in the vicinity may be damaged, survey on housing may be required prior to commencing construction work. The survey extends to details concerning the housing and major positions should be recorded as photographs for use as material in making compensation in the event damage ensues as a result of the construction work.

Moreover, if the extent of the damage is estimated to be large at the planning stage, review should be undertaken with respect to the method of the countermeasure.

(2) Survey on Underground Facilities

Water pipes, gas lines, electrical cables, telephone lines and sewerage are some of the underground facilities.

In designing a road, a system map of the facilities in the relevant section needs to be produced and the type, form, size and depth of such buried structures should be surveyed. In places of particular complexity, excavation with the agencies managing such facilities in attendance should be undertaken in producing the construction plan.

(3) Survey on Surplus Soil

Environmental issues and limitations on disposal area can be expected with respect to the disposal of soil generated as a result of the construction work. For this reason, adequate consideration needs to be given to the disposal of generated surplus soil in foundation work from the planning stage.

Moreover, a disposal plan needs to be made carefully with respect to the large volumes of generates surplus soil in construction of roads in the mountains and in the event such surplus soil is to be disposed in a valley, careful consideration is required from the perspectives of both disaster prevention and the environment.

In recent years, it has become possible to admix lime, cement or other stabilizers to improve the soil properties for use of generated surplus soil as banking or backfill material. For this reason, the existence or the feasibility of installing a soil improvement plant or stockyard, plan for transporting the improved soil and method of using such improved soil effectively should be surveyed and it is desirable to review the effective

use of surplus soil.

(4) Survey on Neighboring Structures

Neighboring important structures targeted for survey may be categorized into ground structures, the main part of which is constructed on the ground such as buildings, rivers, roads, railway facilities and towers and underground structures such as tunnels. Since these structures have possibility of settlement or inclination as a result of change in the ground, loss of balance of soil pressure or decline of ground water level caused by cutting, and possibility of rise or displacement as a result of removal of soil load, adequate surveys are required with respect to important structures and it is desirable to review the structures including measures with respect to the impact of the earth work.

(5) Survey on Construction Conditions

With respect to construction conditions, it is necessary to undertake a survey on the presence or absence of neighboring structures, buried facilities and dangerous substances. Moreover, such limitations as the conditions of the route for transporting construction materials and machinery, restrictions on noise and vibration, construction term, construction process, machinery used and work space as well as methods of temporary drainage during the construction term need to be surveyed.

3. Design

3.1 Introduction

3.1.1 Basic Principles

A road earth structure section must be capable of safely preserving the space required for traffic and of supporting the traffic load conveyed to it via the pavement. Earthwork involves the construction of cut slopes by excavating the ground and of embankments by transporting and compacting excavated soil so that a cutting section and a banking section can be given the important functions mentioned above. Construction of such structures as retaining walls introduced as auxiliary structures in cut slope or embankment work to reinforce an earth structure and culverts buried in the ground to provide a passageway or water channel are usually included in the category of earthwork.

As the existing ground subject to cutting or excavation and the foundation ground above which an embankment is constructed, the soil and rocks used as materials for an embankment are all naturally generated, their properties are very complicated and diverse. As earthwork is conducted under natural weather conditions, it is strongly affected by the seasonal characteristics of the weather, rainfall and seepage water, etc. Accordingly, appropriate measures are required to ensure the sufficient bearing capacity of the foundation ground and subgrade, to maintain the stability of slopes and to control the settlement of embankments so that cut slopes and embankments forming the main body of a road and auxiliary structures of earthwork can be constructed to meet their objectives and to perform their expected functions for many years. As soil is liable to erosion by running water and also to a significant decline of its strength by water infiltration, it is essential to properly deal with surface water and groundwater.

The design of earthwork generally proceeds as described below in correspondence with the road construction stages shown in Fig. 1-2-1.

(1) Outline Design

The outline design is conducted at the planning survey stage and the rough design of such structures as cut slopes, embankments, slope protections, retaining walls and culverts as well as various control/prevention works relating to earthwork for the planned route (and alternative routes, if necessary) is conducted and the rough construction cost is estimated. At this stage, the design work is generally based on existing drawings and reference materials and also on the site reconnaissance results.

(2) Preliminary Design

The preliminary design is conducted at the execution survey stage and the design work is conducted for along the selected route, featuring almost the same items as the outline design. However, as more detailed analysis of each item is required, topographic maps based on aerial photographs and the results of surface geological reconnaissance and sounding, which are conducted at main spots, are used and incorporated in this design. In addition, appropriate design modifications based on a wide consensus are made through consultations with related organizations as needed.

(3) Detailed Design and Cost Estimation

Based on the basic conditions determined by the preliminary design, the detailed structural design is firstly conducted for such structures as cut slopes, embankments, retaining walls and culverts, etc. and also for various control/prevention works. This is followed by decisions on the work execution plan (scheme of haul, determination of borrow pits and spoil areas, calculation of work capacities and scheduling, etc.) following a decision on the work sections. The general design, including temporary structures required for each permanent structure, is then conducted to estimate the construction cost.

3.1.2 Important Points for Design

(1) Verification of Centre Line and Formation Level

The skeleton of the earthwork design, including the heights of cut slopes and embankments and the volume of soil to be handled, is determined by the position of the centre line and the formation level. As these elements are almost decided at the planning survey and execution survey stages, there is little room for modification in the subsequent stages. However, the necessary modifications must be made to the extent permitted by examining the following issues.

(i) In a mountain area where the large-scale excavation of rocks or high retaining walls are required or at a steeply sloping site where large-scale slopes are formed, the position of the centre line and the formation level should be modified as much as possible to reduce the cutting height.

(ii) In the case of a high embankment planned above soft ground, efforts must be made to reduce the banking height as much as possible in view of the stability of the embankment and the settlement of

the embankment over a long period of time. In the case of a low embankment above soft ground with a high groundwater level, it is desirable for the level of the subgrade surface to be at least 50 cm above the highest groundwater level.

(iii) Careful attention should be paid to compatibility between such structures as retaining walls or culverts and earthwork related to cut slopes or embankments, and the suitability of approach works to existing facilities should be carefully analysed.

(2) Temporary Structures

The construction of structures generally requires the construction of temporary structures, including earth retaining, cofferdams and construction roads. As temporary structures are only used during construction work, after which they are removed, they do not attract much attention sometimes. However, temporary structures play an extremely important role in construction work near densely populated areas or construction work in soft ground areas or mountain areas with considerable differences in levels. It must be recognised that temporary structures demand careful attention in regard to safety management, including the prevention of accidents and labour accidents, and also in regard to environmental conservation.

(3) Scheme of Haul

Earthwork involves different types of work, including excavation, transportation, compaction and surface finishing, and the appropriate scheme of haul focusing on construction machinery, transportation distance and soil properties, etc. is required in addition to careful attention to the method to be employed for each type of work in order to ensure rational as well as economical work progress. It has become increasingly difficult in recent years to secure suitable soil spoil areas in and around cities because of stringent environmental demands. Accordingly, it may be necessary to use the surplus soil with stabilisation treatment rather than simply disposing of it as poor soil or its conversion to another use even if long distance transportation is involved. This option often proves to be more economic and less disruptive to the main construction work.

For scheme of haul for large-scale road earthwork, planning of the distribution of materials most suited to each part of an embankment is necessary, taking the properties of the soil produced by cutting work based on the soil investigation results into consideration. For example, the distribution of good quality soil which is easy to compact and which has a large bearing capacity to the upper part of the subgrade or embankment should be planned while placing large rocks and cohesive soil of which the use requires special care to the lower part of the filled up ground.

3.2 Cut Slopes

As natural ground is generally complicated and its properties are not uniform, the rock mass and soil layer forming the existing ground must be classified from the viewpoints of the excavation difficulty and slope stability. Empirically established standard slope gradients are then applied to the classified ground, assuming non-treatment, sodding or simple protective work such as netting, to determine the slope gradient and shape corresponding to the soil and rock properties, and cutting height.

With the passing of time, a cut slope may experience progressive weathering or erosion, resulting in failure due to increased seepage water following a downpour, etc. or major failure at the time of an earthquake. Particularly when a large-scale slope is created on the types of ground described in 3.2.2, special attention is required to prevent an accident or disaster during or after the construction work.

3.2.1 Standard Cross-Section of Cut Slopes

(1) Standard Slope Gradients

Natural ground is extremely complicated and not uniform in its properties, and cut slope tend to gradually become unstable after the completion of work. Therefore, stability analysis is meaningful only in rare cases when examining the stability of cut slope. An overall judgement should be made by fully taking account of the requirements for stability described later by referring to the standard slope gradients listed in Table 3-2-1. Table 3-2-1 indicates the standard values of the gradient of slopes and have been empirically established based upon protection works such as sodding, netting or non-protection. The gradients referred to here are those for the individual slopes not having berms.

The difference between soft and hard rock referred to herein is judged on the basis of the degree of difficulty of excavation (refer to 2-10-3), and is mainly governed by the shearing strength of rock and the amount of rock cracks. The range of standard values shown in Table 3-2-1 is wider than that of standard values for embankments described later, so that determination of gradient of rock slope on the basis of these standard values alone seems to be difficult because there are so many factors involved.

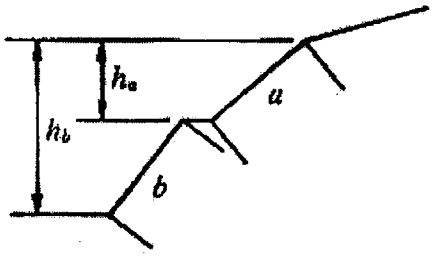
The standard slope gradients shown in Table 3-2-1 may not be applicable in certain cases: (a) cut slope of which the conditions are those described in 3.2.2, (b) higher cut height than those shown in Table 3-2-1 and (c) case where the failure of a cut slope may cause damage to neighbouring structures or requires a long

Table 3-2-1 Standard Gradients of Cut Slope

Soil classification		Cut Slope Height	Gradient
Hard rock			1:0.3 to 1:0.8
Soft rock			1:0.5 to 1:1.2
Sand	Not dense, and poorly graded		1:1.5 to
Sandy soil	Dense	Less than 5m	1:0.8 to 1:1.0
		5 to 10m	1:1.0 to 1:1.2
	Not dense	Less than 5m	1:1.0 to 1:1.2
		5 to 10m	1:1.2 to 1:1.5
Sandy soil mixed with gravel or rock masses	Dense, or well graded	Less than 10m	1:0.8 to 1:1.0
		10 to 15m	1:1.0 to 1:1.2
	Not dense, or poorly grade	Less than 10m	1:1.0 to 1:1.2
		10 to 15m	1:1.2 to 1:1.5
Clayey soil		0 to 15m	1:0.8 to 1:1.2
Clayey soil mixed with rock masses or cobble-stone		Less than 5m	1:1.0 to 1:1.2
		5 to 10m	1:1.2 to 1:1.5

Notes

1) The cut slope height and gradient when a single gradient is not opted for because of the soil composition and other reasons are based on the ideas shown below.



ha : cut slope height for slope surface a

hb : cut slope height for slope surface b

- The gradient does not include a berm.
- The cut slope height vis-à-vis the gradient means the total cut slope height covering the entire cut slopes above the cut slope in question.

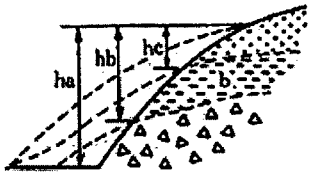
2) Silt is to be classified into the Clayey soil.

3) The table is not applicable to soils not included in the above table.

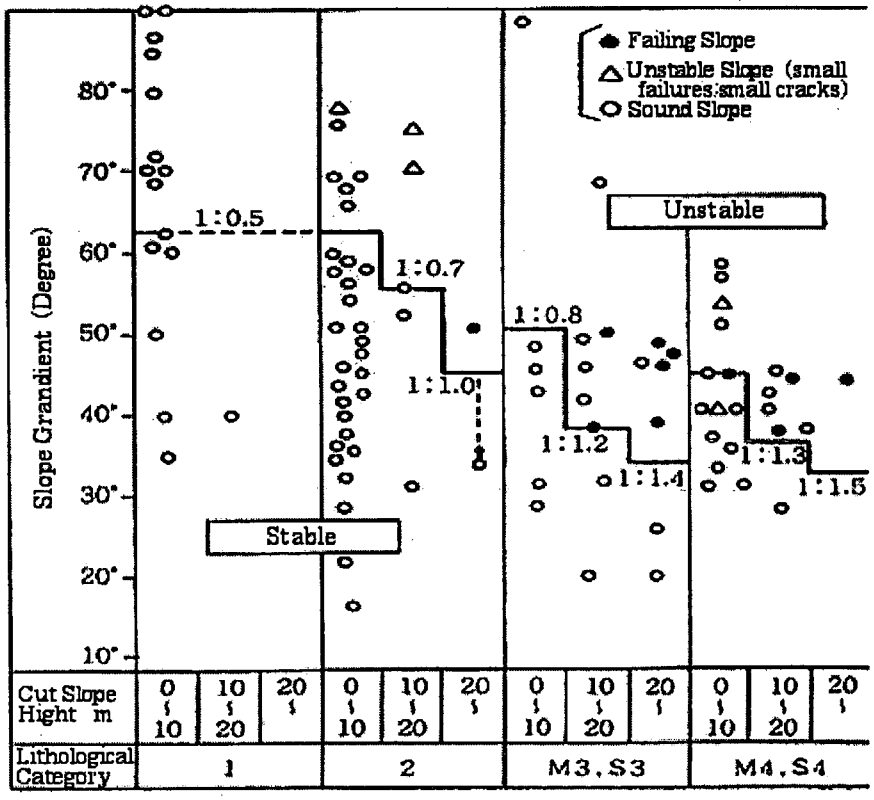
4) In planning of planting for slope, it also takes into consideration the slope gradient suitable for planting.

Table 3-2-2 Standard Slope Gradient for Weathered Granite

Category of Bedrock	Ground Conditions						Slope Height and Gradient (m)
	Conventional Rock Category		Situation of Weathering		Condition of Boring Core	Elastic Wave Velocity of Ground (P Wave)	0 10 20 30 50
Decomposed Weathered Rock	D	D_L D_H	Sedimentary Soft Rock	Decomposed	Sandy	km/s 0.4 – 1.1	1.0 | 1.2 1.2 | 1.5 1.5 | 1.8
Weathered Granite	C	C_L	Extremely Soft Rock	Nearly decomposed rock with few cracks or concentration of cracks	Sandy to fine flakes	1.1 – 1.5	0.6 | 0.8 0.8 | 1.0 1.0 | 1.2 1.2 | 1.5
Slightly Weathered Granite	C	C_M	Soft Rock	Rock discoloured to a yellowish brown to the core and developed joints	Brecciated to short bar-like	1.5 – 2.3	0.4 | 0.5 0.6 | 0.8 0.8 | 1.0 1.0 | 1.2 1.2 | 1.5
Non-Weathered Granite	C	C_H	Hard Rock	Mostly fresh rock mass with developed joints in a massive manner	Bar-like	> 2.3	0.3 | 0.4 0.4 | 0.6 0.6 | 0.8
	B						
	A						



Slope height and rock category
ha: slope height to rock category a
hb: slope height to rock category b
hc: slope height to rock category c



Lithological Classification of Serpentinite

Fig. 3-2-1(1) Lithological Classification of Serpentinite and Proven Slope Gradient

Not Well-Developed Schistocity				Well-Developed Schistocity			
Category	Crack Interval	Appearance	Hammering Result	Category	Crack Interval	Appearance	Hammering Result
1	50 – 10 cm	Olivine is entirely changed to serpentinite but the texture and structure of the olivine still remain. The colour is often dark brown.	Splits along the cracks with ordinary impact. Produces a clear sound to sounding although a slightly murred sound is produced at times.	1	50 – 10 cm	While the distance between the foliating planes is wide, they tightly stick together, resisting foliation.	Splits with ordinary impact. Often splits into masses regardless of the foliating planes.
2	10 – 2 cm crack opening	The original texture is hardly observed. The colour is often brown due to discolouration.	Splits with ordinary impact. Produces a murred sound to sounding.	2	10 – 2 cm		
M3			Easily splits into small pieces.	S3	10 – 2 cm	The distance between the foliating planes is 3 mm – 30 mm and is easy to foliate.	Easily splits into small pieces.
M4		Differentiates to the gravelly part and matrix part by weathering or fracture. The particle size of the matrix is that of sand and clay.	Easily crumbles with some clinging to the tip of the hammer. The rock is fragile and can be broken between the fingers.	S4		The distance between the foliating planes of 1 mm – 5 mm is very narrow. It is clayey and softens with the addition of water.	Differentiates to the gravelly part and matrix part by weathering or fracture. The particle size of the matrix is that of sand and clay.
5	Soft clay with little sand content			5	Soft clay with little sand content		
6	Laterite			6	Laterite		

Fig. 3-2-1(2) Lithological Classification of Serpentinite and Proven Slope Gradient

restoration time, significantly disrupting the road functions. In these cases, it is necessary to change the slope gradient together with the introduction of suitable measures, such as slope protection work and slope drainage work, etc. Slope gradient examples for weathered granite and serpentinite are shown in Table 3-2-2 and Fig. 3-2-1 respectively.

In the case where a cut slope with a steeper gradient than the standard slope gradient is necessary to reduce the earthwork volume, reinforcement of the cut slope with retaining walls and/or the reinforced soil method with steel bar is required.

(2) Slope Shape

As shown in Fig. 3-2-2, the gradient of slope varies depending upon the soils and the rocks, and berms are provided in many cases at transition points where the gradient changes.

A single slope gradient is generally used where the geology and soils are almost the same in the depth direction and in the longitudinal and transverse directions. Where the geology and soils vary considerably and complicatedly, a single gradient of slope suited to the soil of the gentlest gradient may be used even though this is somewhat uneconomical.

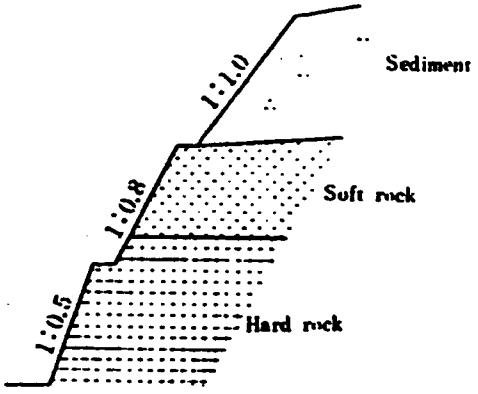


Fig. 3-2-2 Ground Conditions and Shapes of Slopes

(3) Berms

Excepting the case stated in Subclause (2), a berm about 1 to 2m wide will be generally installed in the middle of a cut slope with a large height.

1) Purpose of berm

At the lower portion of a continuous, large slope, the discharge and current speed of the surface water increase, causing the scouring. In this case, the current speed can be reduced by providing a berm in the middle of the slope, or the concentration of the surface water at the lower portion of the slope may be prevented by making a ditch in the berm for draining water outside of the slope. The berm also is used as inspection step or as scaffold for repairing.

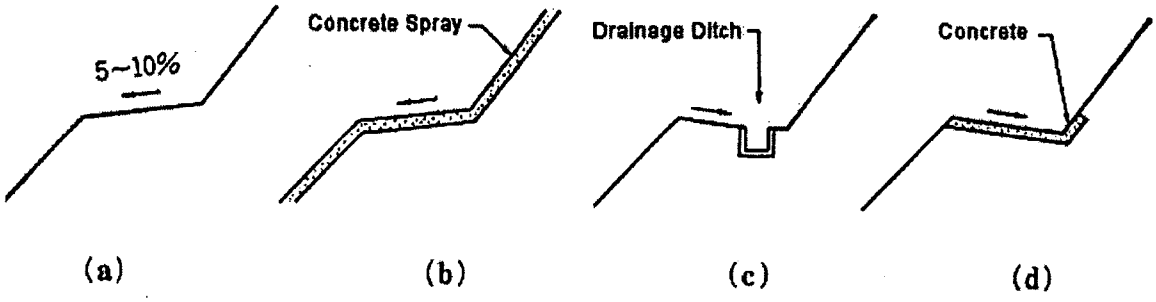


Fig. 3-2-3 Cross- Sectional Gradient of Beam

2) Gradient of berm

Where the drainage facilities are not provided, about 5 to 10% of cross-grade is normally provided for the berm so as to drain water toward the bottom of slope (toe of slope).

However, where the slope is considered to be easily flaked off or eroded, the gradient of the berm should be made in the reverse direction so as to drain water toward the ditch of the berm (refer to Fig. 3-2-3).

3) Location of berms

Therefore, berms should be designed by taking account of difficulty of the inspection and repair, gradient of slope, height of cut, soils of slope, construction cost and other various conditions.

On the cut slopes, berms about 1 to 2 m wide are normally provided every 5 to 10 m of height depending upon the soil, rock and scale of slope.

A wider berm is recommended where the slope is long and large or where the rock fall protection fences are to be installed.

(4) Stability Analysis for Cut Slopes

Stability analysis for cut slopes is not usually conducted except for cut slopes at landslide sites or former failure sites but may be conducted for the design of restoration work for cut slopes which have deformed during or after construction work. In this case, a cross-section of the failure site is used to assume the sliding surface in order to conduct stability analysis against sliding.

3.2.2 Cuts Requiring Extra Precautions

(1) Cuts in Colluvial Deposits or Heavily Weathered Slopes

In the talus cone, weathered slope, trace of volcanic mudflow, or other old slope failure area, slope failure sometimes occurs due to the decline of strength of low solidified soil during the rainfall or earthquake even in the natural condition. If such a place is excavated to a gradient steeper than that of natural slope, the cut slope will become unstable and a failure may sometimes occur such as 'B-1' or 'C-1' in Table 2-2-1.

As countermeasures against this kind of failures, the following methods can be considered:

1) Where a failure such as 'B-1' of Table 2-2-1 is predictable:

As shown in Fig. 3-2-4, if a wide step is provided near the line of bedrock, collapsed sediment and falling from the above will be easily retained on the step. The gradient of the colluvial deposit or portion of weathered layer should be made as gentle as possible.

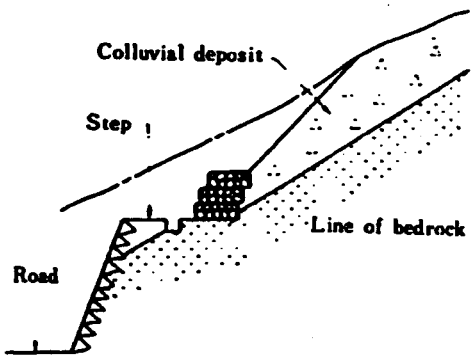


Fig. 3-2-4 Countermeasures against Failures

2) Where a failure such as 'C-1' of Table 2-2-1 is predictable:

In this case, the countermeasures such as large-scale removal of soil (with a slope of 1:1.5 to 2.0 or more gentle, including berms), sufficient groundwater drainage works, or prevention works (pile works against landslide) should be performed.

However, all methods stated above will greatly affect the construction cost and a complete study is required during design.

(2) Cuts in Easily Erodible Ground such as Sandy Soil

Sediments mainly consisting of sandy soil are easily eroded by surface water to frequently result in rock falls, small failure or outflow of soil.

The erosion against water actions should be basically dealt with by slope protection works or drainage works described in Para. 3-4 rather than with the change of slope gradient. Therefore, the water should be fully drained from the top and toe of slope. Permeation of water from the top of slope should be prevented whenever possible. It is important to provide an extra area in front of the toe of the slope so as not to cause any direct influence upon the road surface in the event of failure.

(3) Cuts in Quickly Weatherable Rocks such as Mudstone, Tuff and Serpentinite

If the surface layer of a slope is comprised of tertiary mudstone, shale, tuff with a low degree of solidification, or serpentinite, the surface layer is gradually turned into noncohesive granular sediment due to the release of stresses by excavation, or the repetition of dry-wet thereafter, frequently causing a slope failure 'A-2' and 'A-3' or 'C-3' in Table2-2-1. Because of this, either one of precautions described below should be taken in the design.

1) A stable gradient which will not cause any slope failure in future even after the progress of weathering should be provided. In addition, steps for limiting the damage to a minimum level in the event of slope failure should be provided.
2) Slope should be tightly covered with protection work to restrict the weathering to a minimum.

(4) Cuts in Rocks with Many Fissures

Bedrocks usually have many weak lines such as fault-fractured zones subjected to tectonic motions, and column-shaped or plate-shaped joints created by the contraction during cooling. The former is often seen in the rocks of Mesozoic and Paleozoic formations (schist, gneiss, chert, slate, serpentinite) while the latter is often seen in basalt, andesite, rhyolite and granite.

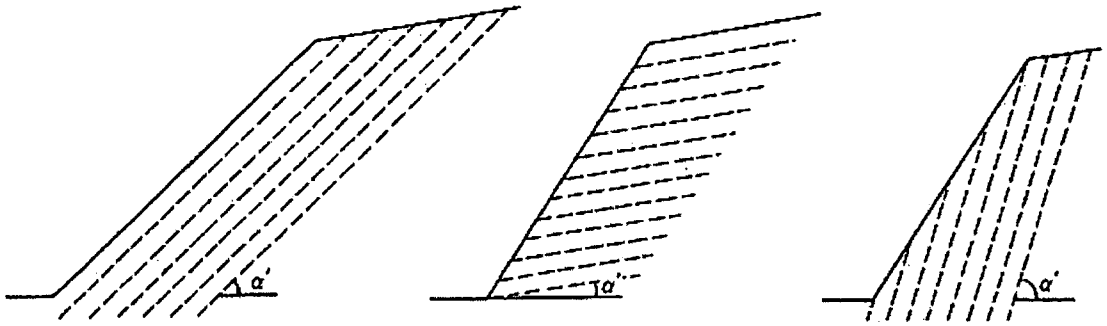
These kinds of rock failures are such as 'B-3' and 'C-2' indicated in Table 2-2-1.

The stability of these slopes is governed by the degree of development of fissures and the degree of fractures, and then an overall review should be performed based upon elastic wave exploration, index of degree of fissuring, and the comparison with the actual records made for existing nearby slopes.

(5) Cuts in Dip Slope Structures with Fissures

Slope failures such as 'B-3' and 'C-2' shown in Table 2-2-1 may sometimes occur where the slope has a dip slope structure and where there are fissures which have developed regularly in a certain direction such as bedding stratification developed in sedimentary rocks, columnar and plate-like joints developed in schist, gneiss or igneous rocks, and where the slant direction of these fissures coincides with the slant direction of the face of the cut slope. In this case, it is desirable to have a gradient of slope (including no berm) equal to or gentler than α' as a rule (refer to Fig. 3-2-5).

In addition, attention is required when joint is developed cross to bedding stratification, even if bedding stratification is support plate, it is easy to cause collapse.



(a) Ordinary Dip Solpe ($30° < \alpha' < 60°$)　(b) Dip Slope with Gentle Gradient ($\alpha' < 30°$)　(c) Dip Slope with Steep Gradient ($60° < \alpha'$)

Fig. 3-2-5 Relationship between Cracks and Slope Surface of Dip Slope

However, if the slant α' of the dip slope structure is gentler than 30 degrees, then the slope is not necessarily considered to be unstable even if the slope ratio is steeper than 1:1.8 and thus the gradient may be determined based upon other factors (Such as degree of development of fissures).

On the other hand, if the dip slope structure has a steep slant α' larger than 60 degrees, the slope is not necessarily considered to be stable in many cases even though the slope ratio is 1:0.6 or less.

Generally, it is not recommended to have a slope steeper than 1:0.8 for slopes higher than 10 m having a dip slope structure.

(6) Cuts where Much Groundwater is Present

Cut slopes tend to become unstable and thus the gradients of such slopes must be gentler if cuts are to be made at places where much spring water is present or groundwater levels are high regardless of geological conditions.

It is required to give the higher priority to the review of groundwater drainage works than the review of gradients of cut slopes in areas where there is much groundwater present.

(7) Large-Scale Slopes

The standard gradient of slopes shown in Table 3.2.1 are for slopes lower than 15 m, and gradients for slopes larger than these must be determined by taking account of the actual conditions. When examining these conditions, the following precautions should be taken:

1) For rocks containing a large amount of montmorillonite, which is considered to be an expansive rock, it is required to secure a gradient of slope capable of assuring the stability even after slight progress of weathering.

2) The cols of mountains frequently have fault-fractured zones and it will become necessary to examine the gradient of a cut slope depending upon the direction and degree of fracture, if these zones are found to be present in deep parts after borings or elastic wave explorations.

3) For the cuts in steep slopes as shown in Fig. 3.2.6, the ground is first classified into soil, soft and hard rock portions and then the cuts are usually made with gradients suited to each portion. However, as shown by Plan A of Fig. 3.2.6 a thin cut layer may be sometimes created up to the top of a slope and an unexpectedly large and long slope may result. If it is required to reduce the area of a cut slope because of restrictions of right-of-way or environmental conditions, it is possible to protect the steep slope by means of prevention work shown by plan B of Fig. 3.2.6 or by other similar structures.

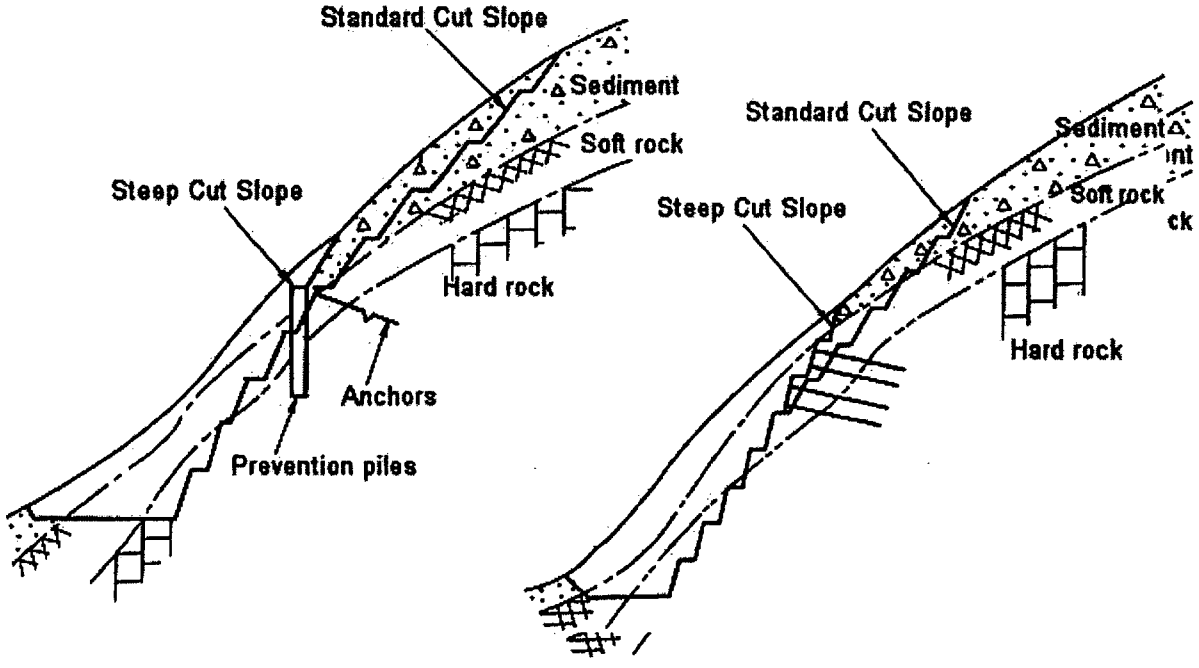


Fig. 3-2-6 Cuts in a Steep Natural Slope

4) In the case of long slopes, it is desirable to provide steps (about 3m wide) for inspection and repair at 20 to 30 m intervals in height in addition to ordinary berms.

(8) Examination of Cut Slope Resistance to Earthquakes

When considering earthquake resistance measures for cut slopes, the relevant measures are not currently thought to be specific measures to prevent earthquake damage but are seen as part of the measures to prevent slopes failures and landslides by causes other than earthquakes. This is because measures to prevent damage by rainwater are believed to act as earthquake resistance measures to a certain extent.

The general trends of the stability of slope protection works at the time of the Great South Hyogo Earthquake are described below for reference purposes.

(i) The following types of slope protection works are believed to have a relatively high level of earthquake resistance.

(a) Structures considering earth pressure (in their design) (retaining walls and others)

(b) Works with close contact to stable ground: works integrated to the ground at the time of an earthquake (cast-in-place concrete cribworks and others which have a certain degree of suppression effect in themselves)

(c) Flexible structures: structures which absorb seismic energy through their deformation (crib retaining walls and others)

(ii) Damaged stone or concrete block masonry works are either dry masonry works or wet stone/concrete block masonry retaining walls with a steep gradient and extremely thin fixing concrete. Higher masonry retaining walls appear to be more likely to be damaged.

(iii) Although mortar concrete spraying works and rockfall prevention nets as well as rockfall protection fences appear to be effective at many sites, their positive effects seem to be limited.

Given the above observation results, the introduction of the types of slope protection works with high earthquake resistance as mentioned above is desirable for those cut slopes with a strong potential to cause serious damage.

3.3 Embankments

As embankments are constructed using soil or rocks with weak strength and proneness to water influence as banking materials, they are liable to settlement or failure. However, unlike cut slopes and others created using natural ground, embankments as rational earth structures meeting specific requirements can be created by selecting suitable banking materials and conducting careful work. The main requirements for embankments are as follows.

(i) Ability to properly bear repeated traffic load without experiencing major deformation or settlement due to traffic load or embankment load

(ii) Sufficient stability against such destructive factors as rainfall, seepage water and earthquakes

There are two types of the settlement of embankments: settlement resulting from compression of the embankment itself and settlement of the foundation ground due to the embankment load and traffic load. Although the rate of compression of an embankment varies depending on the banking materials, banking height and degree of compaction, the rate of compression after open to traffic is approximately within 1% of the banking height in the case of good compaction work. In addition, settlement usually comes to an end after a relatively short period of time. In contrast, the settlement caused by consolidation of foundation ground consisting of organic soil or cohesive soil with high water content is generally severe and tends to continue for a long time after open to traffic. Accordingly, high embankments using compressive soil or those on soft ground require careful analysis at the design stage and measures to control or prevent such settlement must be designed.

3.3.1 Foundation Ground

When the foundation ground is normal ground, i.e. not soft ground, it is safe to assume that there should not be any problem for an embankment. Problems particularly arise with soft ground. When the construction of an embankment on soft ground is planned, consideration of the possibility of embankment failure due to destruction of the foundation ground and of lateral flow as well as consolidation settlement of the foundation ground is essential in accordance with 3.6 – Countermeasures for Soft Ground.

Careful analysis is required even in the case of normal ground as problems may occur in the following cases.

① Particularly high embankment
② Embankment liable to adverse impacts of seepage water from the ground
③ Embankment on inclined ground, such as a hillside
④ Embankment on normal ground which has a thin soft layer at its surface

3.3.2 Banking Materials

The suitability of soil (and its properties) as a banking material can be judged using the contents of Table 3-1-1 as the criteria. The preferred soils to be used as banking materials are those which are easy to work with and which

have such properties as a high shear strength and low compressibility. However, as it is not economically viable to select and use only good quality materials, it is necessary to use available materials which may not be ideal by means of skilful design and work, including the application of stabilization or reinforced earth method with geotextile.

Table 3-3-1 General Evaluation of Soils for Use as Road Materials

Japan Unified Soil Classification	Banking Material	Subgrade Material	Foundation Ground for Embankment	Remarks
(Rock Mass /Cobblestones)	Δ	X	O	Use category depends on the degree of crushing
Gravel (G)	O	O	O	
Gravelly Soil (GF)	O	Δ	O	That containing organic matter or volcanic fine soil (GO, BV) is classified as Δ as both a banking material and the foundations for embankments
Sand (S)	O	O	O	Is classified as Δ as foundation material for embankments above loose, saturated ground
Sandy Soil (SF)	O	Δ	O	That containing organic matter or volcanic fine soil (SO, SV) is classified as Δ as both a banking material and the foundations for embankments
Silt (M)	Δ	Δ	Δ	
Clayey Soil (C)	Δ	Δ	Δ	
Volcanic Cohesive Soil (V)	Δ	Δ	Δ	
Organic Soil (O)	Δ	X	Δ	
Highly Organic Soil (Pt)	X	X	Δ	

O: virtually no problem; △: use with care or in need of some treatment; X: should not be used
Note: (Rock mass/cobblestones) is not part of the Japan Unified Soil Classification.

When more than one type of soil, for example, gravelly soil and cohesive soil, are available on site for use as banking materials, their differential use in the following manner to suit their individual characteristics is desirable, taking the stability of the embankment and pavement proper into consideration.

(i) Case with No Problem for Embankment Stability

Even if there is no problem in regard to the stability of the embankment, good quality materials, such as gravelly soil and sand, should be used in the upper part of the embankment near the pavement.

(ii) Case with Problem for Embankment Stability

In addition to the care taken in case (i) above, gravelly soil or sand with a low fine grain content should be used in the lower part of the embankment as much as possible when there is a risk of the inflow of spring water to the embankment because of soft ground, inclined ground or valley so that a potential rise of the pore water pressure inside the embankment can be prevented to reduce the risk of embankment failure.

Embankment slopes may be liable to erosion by surface flow depending on the properties of the banking materials and it is necessary to cover the slope surface with such erosion-proof top soil as clay (C) or gravelly soil (GF) in some cases (refer to Fig. 3-3-1). It is commonly said that such top soil must have a vertical thickness of 30 cm. A top soil thickness of 2 – 3 m is desirable for actual earthwork to allow horizontal thin layer rolling compaction by the machinery used to compact the filled up ground.

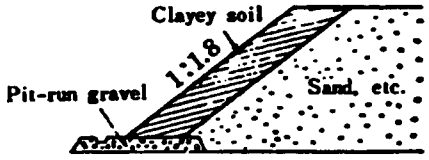


Fig. 3-3-1 Covering for Slope of Embankment

3.3.3 Compaction of Embankments

When soil is compacted using such compacting machinery as a roller or rammer in the laboratory compaction test, soil undergoes transformation of arrangement while being compressed and crushed. As small particles fill the voids between larger particles, soil becomes more and more compacted. The state of compacted soil not only varies depending on the type of soil and the water content level but is also considerably affected by the specific compaction method used and the level of compacting energy.

The water content where the void ratio of soil compacted by a specific compaction method and the compaction energy become the smallest, i.e. the maximum dry density, is called the optimum moisture content (w_{opt}) under specific compaction conditions. The corresponding dry density is called the maximum dry density ($\rho_{d\max}$) (see Fig. 3-3-2).

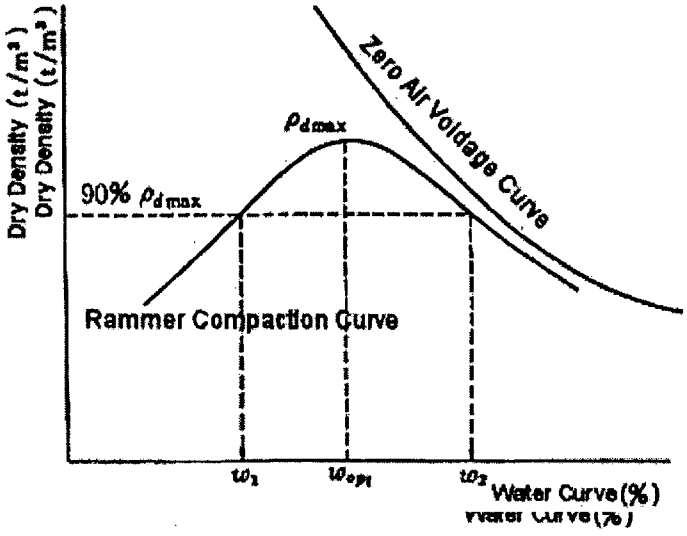


Fig. 3-3-2 Soil Compaction Curve

Soil compacted to the optimum moisture content and maximum dry density has, by definition, the minimum void ratio under the compaction conditions in question and the amount of air remaining in the voids can no longer be removed by the applied compaction energy. This means that the soil properties hardly change due to seepage after compaction as the remaining voids can hardly absorb water.

Meanwhile, the strength of compacted soil changes depending on the soil particles or water content. However, it is generally agreed that, immediately after compaction, the maximum strength and deformation resistance and the minimum compressibility are achieved at water content which is slightly lower than the optimum moisture content. This is particularly noticeable in the case of fine grained soil of which the properties are substantially affected by the water content level. When this soil is inundated, it begins to swell because of the relatively larger air voids than those under the optimum moisture content and maximum dry density. As a result, it is common for the excellent strength of this soil to decline to the lower level than strength achieved under the optimum moisture content and maximum dry density conditions. In contrast, soil which is compacted at a water content level which is slightly higher than the optimum moisture content has few air voids which can be replaced by water due to water absorption. Because of this, it is liable to little change of its strength due to inundation but its strength is inferior to that of soil compacted to the maximum dry density.

As it is necessary to anticipate inundation during the service period of road embankments, the durability of the properties of the compacted soil when inundated is the principal requirement. For this purpose, the state of optimum moisture content and maximum dry density described above is desirable.

For the compaction of an embankment, it is essential to pre-determine the target degree of compaction, water content during construction and spreading depth of the banking materials, etc. In general, when sandy soil or gravelly soil is used as the banking material, it is a common practice to regulate the degree of compaction by the density. The required regulation for filled up ground is that the field dry density after compaction should be 85 – 90% of the maximum dry density achievable by Method A for the laboratory test for soil compaction using a rammer as stipulated by JIS A1210.

In regard to the subgrade, this value is commonly regulated to above 90 – 95%.

When the banking material is gravelly soil not containing fine particles, it is little affected by changes of the water content. Accordingly, it is unnecessary to pay constant attention to the water content during construction and it is sufficient to concentrate on achieving the regulated dry density value. Even with sandy or gravelly soil, however, when its fine particle content increases, its workability and post-compaction strength are more liable to impacts by

the water content. It is, therefore, essential to properly observe the regulated water content during construction, targeting the optimum moisture content established as the reference value by the compaction test.

The importance of observing the regulated water content during construction together with the regulated degree of compaction when using fine grained soil as the banking material has already been described above. The natural water content of cohesive soil, typically volcanic cohesive soil, tends to be much higher than the optimum moisture content determined by the reference degree of compaction under the JIS and it is often difficult to achieve the regulated degree of compaction, i.e. water content during construction > w_2 in Fig. 3-3-2, if the natural water content is selected as the water content during construction. When looking at the stability of the soil properties against post-compaction inundation, it should be sufficient for the degree of saturation and the air void ratio to fall within the range of 85% to 95% for the former and 2% to 10% for the latter in the case of cohesive soil. In some cases, the degree of compaction is accordingly regulated within the above range while the upper limit for the water content during construction is determined based on the relevant necessity suggested by the strength and deformation characteristics of the compacted soil. In these cases, the upper limit for the water content during construction tends to be selected with reference to the minimum required strength to secure trafficability during construction. Standard target value for soil compaction is shown in Table 3-3-2.

Table 3-3-2 Standard Target Value for Compaction

Subgrade	Filled up Ground	Clayey soil
Degree of compaction more than 90～95%	Degree of compaction more than 85～90%	Degree of saturation 85～95% Air void ratio 10～ 2%

3.3.4 Standard Cross-Section of Embankments

Embankments should be rationally designed, taking the site conditions (ground conditions, banking materials and weather conditions, etc.), embankment stability and construction conditions into consideration and at least slopes between neighbouring berms should have a uniform gradient.

The empirically determined standard values shown in Table 3-3-3 and Fig. 3-3-3 are commonly used as the standard slope gradients for embankments in correspondence with the specific banking materials and banking height. The standard slope gradients shown in Table 3-3-3 are the maximum grades required to secure the stability of embankment slopes which enjoy sufficient bearing capacity of the foundation ground, have no risk of groundwater infiltration or inundation from the foundation ground, have undergone horizontal thin layer rolling compaction and have also received erosion control measures (such protective works as soil cover, sodding, simple slope cribwork and concrete block pitching, etc.), if necessary.

Table 3-3-3 Standard Gradients of Slopes Corresponding to Banking Materials and Heights of Embankments

Banking materials	Height of embankment (m)	Standard gradient	Remarks
Well-graded Sand (S), gravel, and sand mixed with gravel (G)	Less than 5m	1:1.5 to 1: .8	To be applied to embankments with sufficient bearing capacity at foundation ground, which is not affected by inundation. Typical unified soil classification are shown in ( ) for reference. In case of exception of standard slope is needed the stability calculation.
	5m to 15m	1:1.8 to 1:2.0	
Poorly-graded Sand (SG)	Less than 10m	1:1.8 to 1:2.0	
Rock masses (including muck)	Less than 10m	1:1.5 to 1:1.8	
	10m to 20m	1:1.8 to 1:2.0	
Sandy soil (SF), hard clayey soils and hard clay (hard clayey soils and clay of alluvium, loam, etc.)	Less than 5m	1:1.5 to 1:1.8	
	5m to 10m	1:1.8 to 1:2.0	
Volcanic cohesive soils (V)	Less than 5m	1:1.8 to 1:2.0	

Note: Height of embankment is to be measured from toe to top of slope.

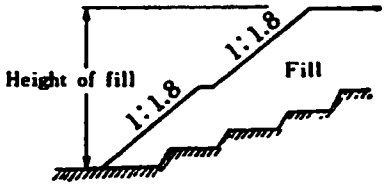


Fig. 3-3-3 Height of Embankment

When road embankments are also used as river or coastal dykes, proper attention must be paid to the slope gradient and the prevention of erosion due to scouring, etc.

In general, the introduction of berms of approximately 1 – 2 m in width for every 5 – 7 m drop in height from the top of the slope is desirable. The basic idea for the introduction of berms is described below.

(i) Berms can function as step for maintenance and scaffolding for repair work, if necessary (place to accommodate the foundations for additional slope protection structures).

(ii) Berms can slow down the velocity of surface flowing rainwater to prevent erosion during and after construction work. For this purpose, a drainage ditch is often introduced for embankment berms, excepting low embankments.

3.3.5 Stability of Embankments

In general, while the standard slope gradients shown in Table 3-3-3 are used as the slope gradients for embankments, stability analysis is required in the cases described below. In these cases, however, decisions on the slope gradient and embankment structure primarily based on the stability analysis results should be avoided as it is important to make a comprehensive judgement after full investigation of the relevant work records for neighbouring areas and examples of past disasters involving similar soil conditions.

(1) Embankments Requiring Stability Examination

Since the standard gradients set forth in Table 3-3-3 cannot be applied under the conditions stated below, the stability analysis should be performed and then the embankment structure including the selection of banking material, subsurface drainage ditches, gradient of slope and protection work should be properly designed.

1) Conditions of embankment itself

- Where the height of embankment exceeds the standard values shown in Table 3-3-3.
- Where the water content in the banking material is high, and the banking materials consist of soils in which pore water pressure tends to increase easily. (Such as soil with a high clay, silt fraction, volcanic ash fall deposit)

2) External conditions

- Where the foundation ground is unstable, such as soft ground or landslide area.
- Where the embankment is easily affected by spring water from the ground (such as half-bank and half-cut section, widening of embankment, embankment on slope, embankment across valley).
- Where the embankment slope may be inundated or the toe of the slope may be eroded during floods (such as embankment on ponds, or along river).

3) Effect of embankment failure

- Where serious damage may be given to the adjacent structures in the event of failure or there may be no access due to no detour.
- Where a long time is needed for restoration work and the functions of road may be considerably disturbed in the event of failure (such as embankment on inclined ground like mountainous road and where there is no other alternate road).

(2) Stability Analysis for Embankments

(i) Examination of Stability in Ordinary Condition

Examination of stability in ordinary condition is conducted on the following two cases.

- Immediately after the completion of embankment construction work.
- After the completion of embankment, whenever the embankment stability is questioned due to the impacts of rainwater or seepage water from a mountain area.

If the stability of the foundation ground for the embankment is a problem, refer to 3.6 – Countermeasures for Soft Ground. An embankment immediately after its construction may become unstable as the increased pore water pressure due to the increased weight of the embankment itself remains in the embankment. Even if the pore water pressure during construction is zero, subsequent rain or groundwater seepage from a mountain area could destabilise the embankment, necessitating examination of these possibilities.

(a) Expression for Stability Analysis

For stability calculation, the slice method assuming the circular sliding surface shown in Fig. 3-3-4 is commonly used. With this method, the soil mass on the sliding surface is divided into several slices and the shearing force and resistance force at each slice are calculated and totalled to establish the safety factor based on the relative ratios of the total shearing force and resistance force. The number of slices is normally more than 6 or 7.

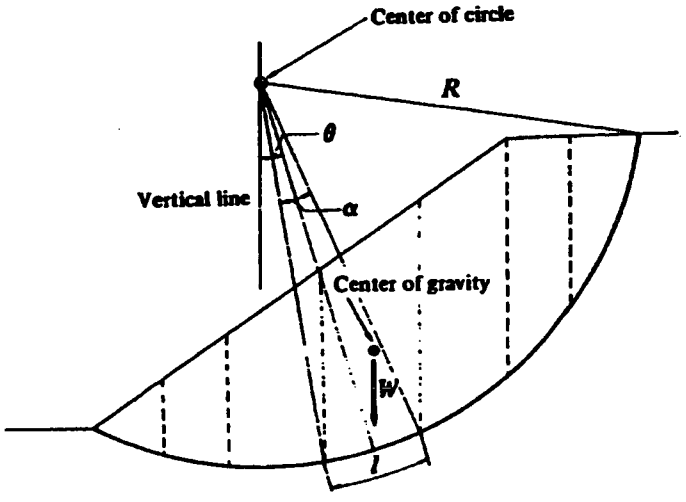


Fig. 3-3-4 Calculation of the Stability for Circular Sliding Surface

Depending on the structure of an embankment, another expression is used, assuming a compounded sliding surface, including straight lines, instead of a circular sliding surface.

$$Fs = \frac{\sum \{c \times \ell + (W - u \times b)\cos\alpha \times \tan\phi\}}{\sum W \times \sin\alpha} \qquad \text{..... (3.3.1)}$$

Where,

F_s : safety factor

c : cohesion (kN/m^2 (tf/m^2))

ϕ : shearing resistance angle

l : length of sliding surface sliced

W : total weight of slices (kN/m (tf/m))

u : pore water pressure (kN/m^2 (tf/m^2))

b : width of slice (m)

α : angle formed by (i) the straight line linking the centre point of the sliced sliding surface and the centre point of the sliding surface and (ii) the plumb line (degree)

The yardstick for embankment design is to make the minimum safety factor in ordinary condition to be 1.2 or higher. Fig. 3-3-5 shows an example of calculating the minimum safety factor.

There are two kinds of stability analysis: total stress analysis and effective stress analysis.

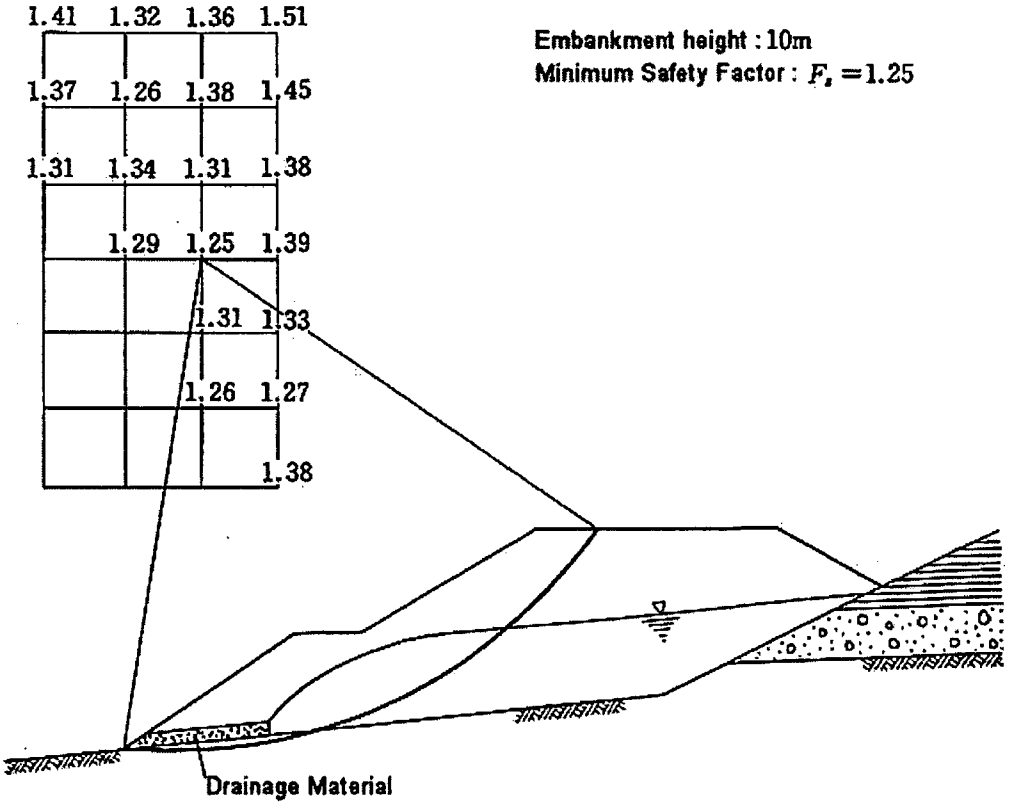


Fig. 3-3-5 Example for Minimum Safety Factor Calculation

(b) Calculation of Strength Parameters

The shear strength parameters of a banking material are calculated by the triaxial compression test under the confining condition corresponding to the height of an embankment. For this test, a specimen

conforming to the working conditions (in terms of the water content and degree of compaction) is prepared from the planning banking material.

< Embankment Stability Analysis Immediately After Construction >

- Total Stress Analysis

a. Embankments of up to 5 m in height and made of cohesive soil with a low permeability coefficient ... UU test

A strength increase due to consolidation is not expected for embankments of up to 5 m in height.

b. Embankments of higher than 5 m and made of cohesive soil with a low permeability coefficient ... CU test

The undrained shear strength corresponding to the consolidation stress immediately after construction is adopted.

c. Embankments with a high permeability coefficient .... CD test

- Effective Stress Analysis

d. Embankments made of cohesive soil with a low permeability coefficient .... CU test

e. Embankments with a high permeability coefficient .... CD test

< Stability Analysis for Embankments long after Completion>

- Total Stress Analysis

f. Embankments made of cohesive soil with a low permeability coefficient .... CU test

g. Embankments with a high permeability coefficient .... CD test

- Effective Stress Analysis

h. Embankments made of cohesive soil with a low permeability coefficient .... CU test

i. Embankments with a high permeability coefficient .... CD test

It is inferred that the long-term stability of embankment slopes is generally reduced by an increase of the pore water pressure inside the embankment. Therefore, the effective stress method is believed to produce a more accurate result when evaluation is based on actual measurement values of the pore water pressure. However, the total stress method is more convenient if there are no actual measurement values of the pore water pressure.

(c) Classification of Pore Water Pressure and Calculation Methods

The difference between the effective stress analysis and the total stress analysis in determining the pore water pressure is that it may be necessary to evaluate the pore water pressure caused by sliding (u_A) for the former in certain cases while it is unnecessary to consider the u_A for the latter. The pore water pressure (u) used in the stability analysis expression is generally the sum of several components.

$$u = u_A + u_r + u_0 \qquad \text{..... (3.3.2)}$$

u_A : Excess pore water pressure produced by sliding

This pore water pressure can be determined by the undrained laboratory triaxial test which fully reproduces the on-site stress conditions.

u_r : Excess pore water pressure produced by compression during construction due to the increased weight of an embankment

The pore water pressure during and immediately after embankment construction work is estimated by firstly calculating the pore water pressure under the undrained condition (maximum pore water pressure) based on the standard consolidation test results with the banking materials compacted with the water content during construction and then estimating the degree of pressure reduction of the pore water drained during the construction period passed on the basis of the degree of consolidation.

u_0 : Pore water pressure produced by seepage water from the lower part or side of an embankment or the seepage of rainwater

This pore water pressure is estimated by the graphical method, etc., taking the soil properties, shape, location of the drainage layer(s) and conditions of the existing ground, etc. into consideration.

(d) Stability Analysis for Failed Slopes

In general, undisturbed samples are collected near the sliding surface of a failed slope and their shearing test results are used to determine the strength parameters for stability analysis. However, given the few cases where the soil test on failed soil is possible and the little time allowed for restoration work, it is rational to use the method which infers the failed sliding surface and to calculate the average value of the shear strength along the entire sliding surface.

To be more precise, the shear strength of a failed slope can be estimated by the following expression for its use as reference information for the examination of feasible countermeasure.

$$\tan\phi = \frac{\sum (W \times \sin\alpha - c \times \ell)}{\sum \{(W - u \times b) \times \cos\alpha\}} \qquad \text{..... (3.3.3)}$$

Where,

c : cohesion (kN/m^2 (tf/m^2))

ϕ : shearing resistance angle (degrees)

Here, the pore water pressure u is estimated, taking the situation of spring water at the time of failure and the original topography, etc. into consideration. The shear strength parameter ϕ ϕ is determined by preparing a $\tan\phi - c$ diagramme based on the arbitrary selection of the cohesion c in the approximate range of zero to 10 kN/m^2 (0 – 1 tf/m^2).

(ii) Examination of Stability at Time of Earthquake

Embankments damaged by large earthquakes in the past were often those which were constructed above loose sandy ground prone to liquefaction or those which were constructed above inclined ground where embankments above soft clayey ground were also occasionally damaged. The descriptions given in 3.10 – Retaining Walls may be referred to regarding the level of design earthquake motion which must be considered for examination of the stability at the time of an earthquake.

Stability analysis is also conducted as a method of securing the required aseismicity of an embankment. As the stability analysis for embankments is not fully established as described earlier, careful construction work based on technological experience in the past may be considered as an alternative. Among the various methods to examine the stability of an embankment at the time of an earthquake, the simplest way is based on the seismic coefficient method assuming a circular sliding surface. Research efforts have recently been in progress on the method using the residual strength and the method using the elasto-plastic finite element method, etc. in addition to the methods described below to examine the stability of embankments on flat land when the foundation ground is liquefied. These new methods can be used for detailed examination and other purposes, if necessary.

(a) Case of Medium-Scale Earthquake Motion

The following expressions can be used to calculate the safety factor at the time of an earthquake as stability analysis based on the seismic coefficient method assuming a circular sliding surface (Fig. 3-3-6).

$$Fs = \frac{\sum\left[c \times \ell + \left\{(W - u \times b) \times \cos\alpha - k_h \times W \times \sin\alpha\right\}\tan\phi\right]}{\sum(W \times \sin\alpha + h/r \times k_h \times W)} \quad \cdots\cdots (3.3.4)$$

Where,

F_s : safety factor

r : radius of sliding surface

c : cohesion (kN/m^2 (tf/m^2))

l : arc length of sliced sliding surface (m)

W : weight of slice (kN/m (tf/m))

u : pore water pressure (kN/m^2 (tf/m^2))

b : width of slice (m)

α : angle formed by (i) the straight line linking the centre point of the sliced sliding surface and the centre point of the sliding surface and (ii) the plumb line (degrees)

k_h : design lateral seismic coefficient (dimensionless)

ϕ : shearing resistance angle (degrees)

h : vertical distance between the centre of gravity of each slice and the centre point of the sliding surface (m)

$$Fs = \frac{\sum(\tau_f \times \ell)}{\sum(W \times \sin\alpha + h/r \times k_h \times W)} \qquad \text{..... (3.3.5)}$$

Where,

τ_f : dynamic strength of soil expressed in total stress (kN/m^2 (tf/m^2))

For seismic design of embankment, the minimum safety factor should be 1.0 or higher. As an earthquake seldom occurs together with a downpour, it is unnecessary to consider seepage water originating from a downpour for embankment design in preparation for an earthquake.

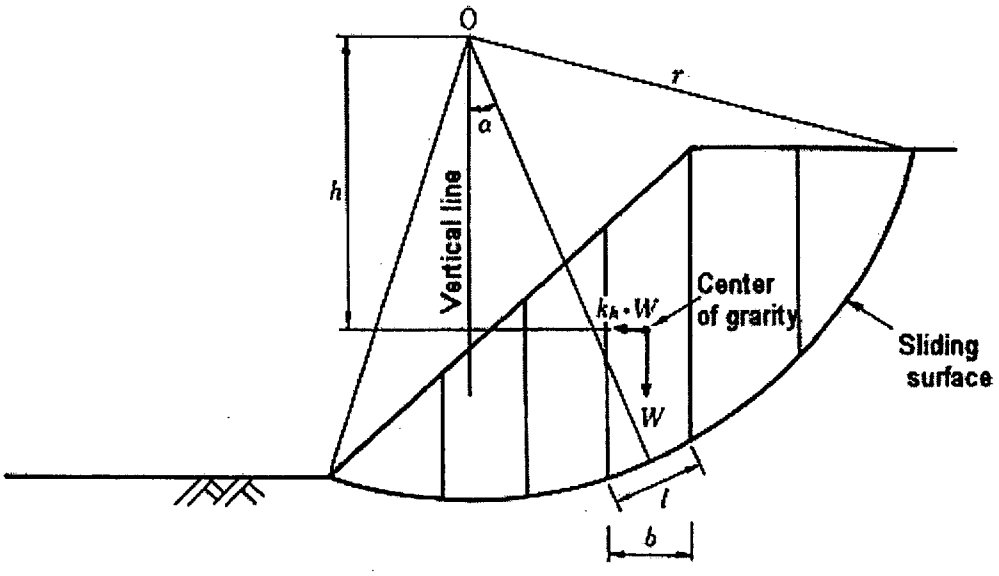


Fig. 3-3-6 Calculation of Seismic Stability for Circular Sliding Surface

(b) Case of Large-Scale Earthquake Motion

In this case, the use of a method which can evaluate the degree of settlement and deformation of the embankment at the time of an earthquake is desirable. However, there is currently no reliable method

which can accurately evaluate the degree of settlement and the stability of an embankment at the time of a large-scale earthquake. One tentative idea is the use of Expression (3.3.5). In this case, the dynamic strength of the soil is calculated based on the failure strain criteria at some 10 – 15%.

(3) Stabilisation of Banking Materials

Cohesive soil with a high water content, including volcanic cohesive soil, is not particularly suitable for high embankments because it becomes soft when kneaded by construction machinery and also because it is characterised by low strength and high compressibility during and immediately after construction.

In the case where the only available material nearby is poor cohesive soil and the use of good quality material poses an economic problem because of the high transportation cost, stabilization of soil is required to improve the properties of the poor soil for its use as a banking material. The common stabilisation method for banking materials is to admix lime as a stabiliser to reduce the water content of the material coupled with an increase of the soil strength by the binding action of the stabiliser. The yardstick for the admix volume of lime is shown in Table 3-3-4. However, it is a good idea to decide the volume by conducting the mix proportion test corresponding to the construction conditions.

Stabilisation by lime is also effective to improve the trafficability of the hauling road for banking materials. Meanwhile, when cement is used as the stabiliser, cement is admixed to the banking material at a rate of 2 – 10% (usually 2.5 – 4%). This admixture is then compacted so that the binding as well as hardening action of the cement can increase the strength and bearing capacity of the banking material. In general, cement as a stabiliser is used for silty soil and sandy soil with a high proportion of fine sand.

When the subgrade material is characterised by a high water content and high plasticity (cohesive soil) or insufficient strength (for example, sandy soil with a high proportion of fine sand), stabilisation using lime or cement as the stabiliser often proves to be effective.

Table 3-3-4 Relationship between Soil and Added Amount of Lime

Type of Soil	Type of Lime	Test Blending Ratio	Remarks
Clay or soil with high clay content	Hydrated lime	5 – 15%	▪ Highly effective with montmorillonite-type clay ▪ 15 – 25% may be required for Kanto loam ▪ Quick lime is effective for clayey soil with a high water content
	Quick lime	3 – 15%	
Silty soil	Hydrated lime	5 – 10%	▪ Effective if an appropriate quantity of clay is contained

Note: The test mixing ratio means the percentage in the dry weight of soil of not more than 5 mm.

(4) Reinforced Embankment Method

Recent years have seen an increasing number of steep slope embankments, the stability of which is enhanced by the application of reinforcing materials, including geotextiles. There are many ways to apply geotextiles to embankments as listed below.

(i) Prevention of shallow slippage failure or surface erosion

(i)-a Assistance for rolling compaction near the slope surface

(i)-b Prevention of the surface erosion of an embankment made of soil prone to erosion (fine sand, etc.)

(i)-c Prevention of the surface slip of an embankment made of a material such as sandy soil which has a certain degree of permeability and which is liable to the significant decline of its strength due to an increased degree of saturation

(ii) Prevention of deep slippage failure

(ii)-a Dissipation of the excess pore water pressure during the construction of an embankment using cohesive soil with a high water content

(ii)-b Drainage and reinforcement of embankments (half-bank and half-cut, widened embankments, embankments on inclined foundations and embankments across valleys) which are prone to the impacts of seepage water from the ground

(iii) Construction of stable high embankments

(iv) Construction of stable steep slope embankments

(v) Improvement of the stability of important banking structures

In the case of (i)-a, the laying of a geotextile, of which the shear strength is 2 kN/m (200 kgf/m) or higher, as a rolling compaction facilitation material at intervals of 30 cm and lengths of 2 m is recommended.

In the case of (i)-b, the laying of a geotextile, of which the shear strength is 2 kN/m (200 kgf/m) or higher, as an erosion prevention material at intervals of approximately 30 m and lengths of 1 m is recommended.

In the case of (ii)-a, a geotextile capable of draining can be used to reinforce an embankment by means of dissipating the excess pore water pressure during embankment construction using cohesive soil with a high water content and facilitating an increase of the soil strength through consolidation. In such cases, a combination of the tensile reinforcement effect and drainage effect of the geotextile is often expected to take place.

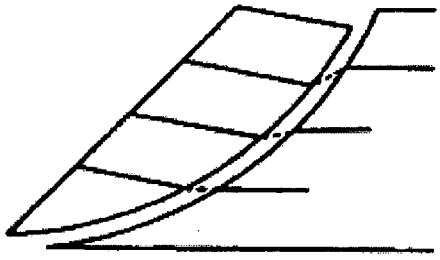
In the cases of (i)-c, (iii), (iv) and (v), the following design principles are adopted in order for the geotextile to perform its tensile reinforcement effect.

The following three failure modes of reinforced embankment are considered and the required tensile strength, interval and length of the geotextile are determined to ensure the embankment stability for each mode of failure (see Fig. 3-3-7).

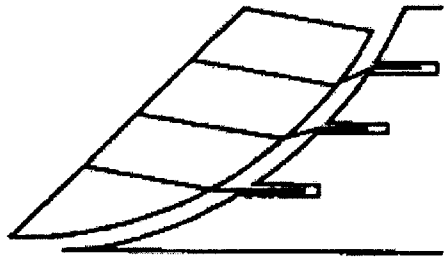
(i) Examination of slippage failure inside the reinforced area (analysis of internal stability)

(ii) Examination of slippage failure outside the reinforced area (analysis of external stability)

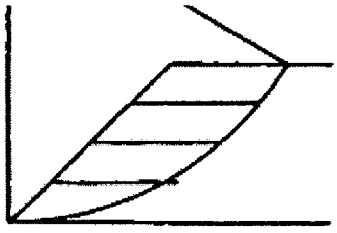
(iii) Examination of falling and erosion, etc. of the soil mass held between the geotextiles (analysis of partial stability)



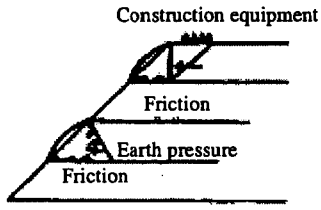
(a) Examination of failure and elongation for geotextile



(b) Examination of pull-out of geotextile



(c) Examination of sliding failure outside the reinforced area



(d) Examination of falling of soil or facilitation of rolling compaction for slope

Fig.3-3-7 Failure mode for reinforced embankment

The general design procedure is shown in Fig. 3-3-8. Firstly, stability analysis is conducted for an unreinforced embankment. If the resulting safety factor fails to satisfy the required value ($F_s \geq 1.2$), reinforcement using geotextiles is considered.

Next, the required tensile force of the geotextile is calculated based on examination of the internal stability. For this purpose, one of the following expressions is used (see Fig. 3-3-9).

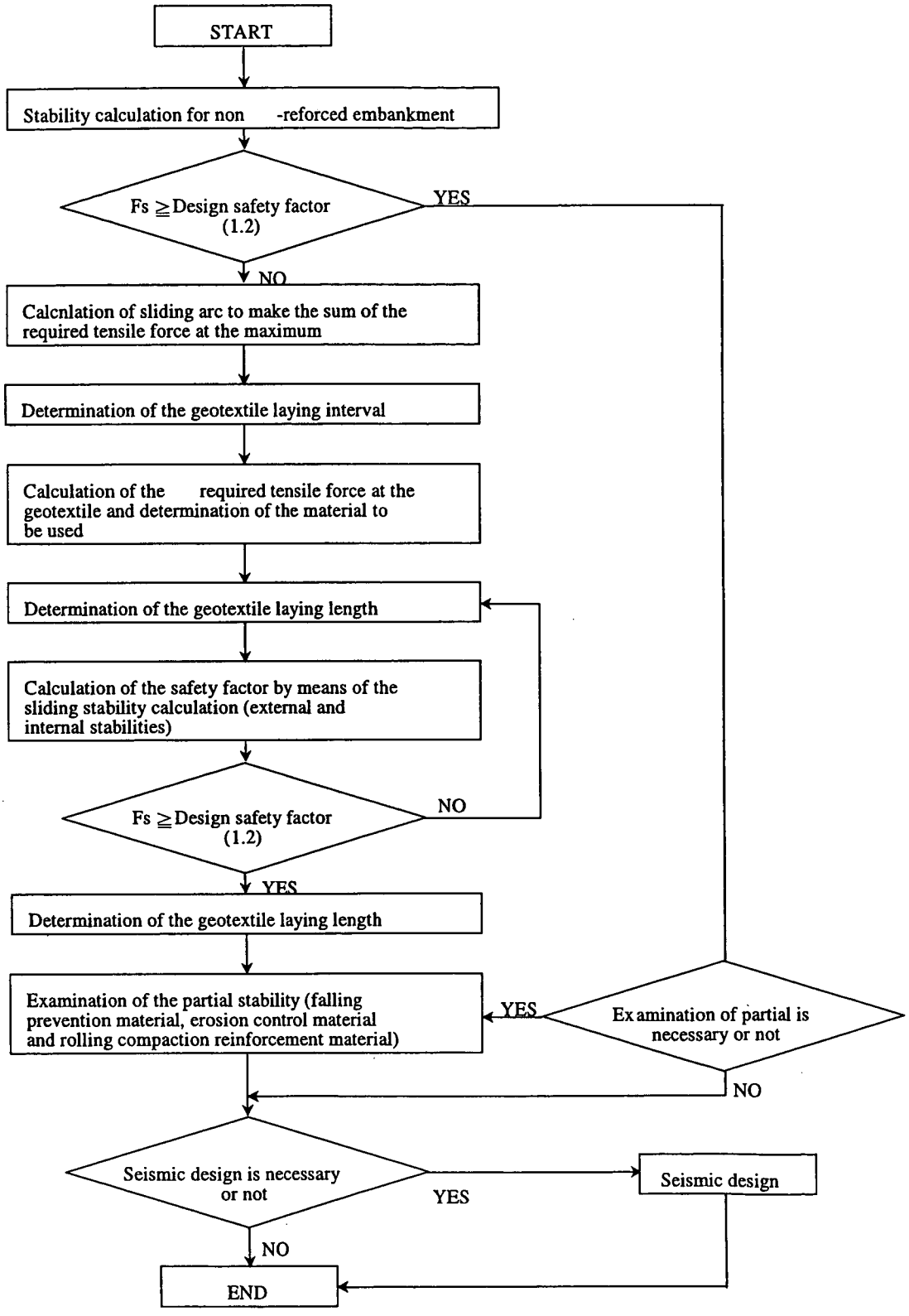


Fig. 3-3-8 Standard Design Method for Reinforced Embankment

$$Fs = \frac{M_R + \Delta M_R}{M_D} = \frac{M_R + R\sum Ti}{M_D} \quad \text{..... (3.3.7)}$$

$$Fs = \frac{M_R + \sum R(Ti \times \cos\alpha + Ti \times \sin\alpha \times \tan\phi)}{M_D} \quad \text{..... (3.3.8)}$$

Where,

M_R : resisting moment of soil mass in unreinforced embankment

($M_R = R\sum\{c \cdot l + (W - u \cdot b)\cos\alpha \cdot \tan\phi\}$, u : pore water pressure)

M_D : sliding moment of soil mass in unreinforced embankment

($M_D = R\sum W \cdot \sin\alpha$)

ΔM_R : increment to resisting moment by tensile force of geotextile

T_i : tensile force of each geotextile

R : radius of sliding circular arc

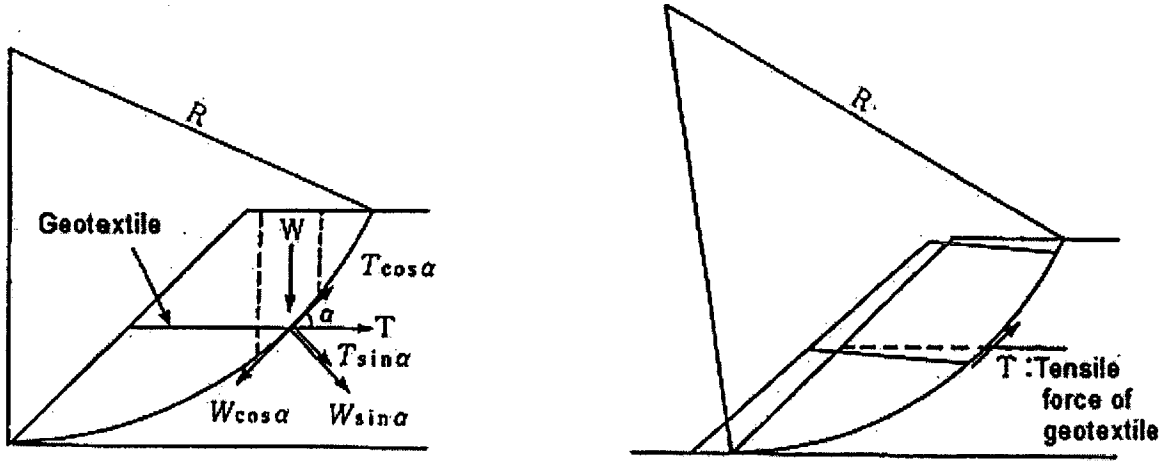


Fig. 3-3-9 Basic Idea of Force of Reinforcement in Internal Stability Calculation for Reinforced Embankment

The required tensile force (T_{req}) per geotextile is then calculated by Expression (3.3.9) and a geotextile with the design tensile strength to meet the required tensile force is selected.

$$T_A \geq T_{req} = \sum T_{req} / N = (\sum T_i)_{\max} / N \quad \text{..... (3.3.9)}$$

Where,

$\sum T_{req}$: total of required tensile force of geotextiles

N : number of geotextiles laid

The design tensile strength T_A of the geotextiles here takes the level of strain as well as the creep ultimate strength of the geotextiles into consideration instead of simply using the maximum tensile strength established by the tensile test. Moreover, the necessary safety factor must be adopted when geotextiles are damaged during construction or are bonded for use or are used under strong acid or alkaline soil conditions.

Meanwhile, the laying length of the geotextile should be determined to secure a sufficient length beyond the potential sliding surface described above.

The pull-out resistance force of the geotextile is calculated by one of the following expressions.

$$T_p = 2(\alpha_1 c + \alpha_2 \sigma_v \tan\phi) L_e / F_s \quad \text{..... (3.3.10)}$$

$$T_p = 2(c^* + \sigma_v \tan\phi^*) L_e / F_s \quad \text{..... (3.3.11)}$$

Where,

T_p : pull-out resistance force

L_e : fixed length of geotextile

F_s : safety factor relating to pull-out resistance (usually $F_s = 2.0$)

c, ϕ : cohesion and shearing resistance angle of soil

σ_v : vertical stress at the laying location of geotextile

α_1, α_2 : revision coefficient relating to friction between the soil and geotextile

($\alpha_1 = 0, \alpha_2 = 1.0$ for sand, $\alpha_1 = 0.5, \alpha_2 = 1.0$ for cohesive soil or sandy soil)

c^*, ϕ^* : apparent cohesion and shearing resistance angle of the soil and geotextile established by the pull-out test, etc.

The next stage is to examine the internal and external stability of the layout of geotextiles designed in the above manner using Expression (3.3.7) or Expression (3.3.8) to check whether or not the calculated safety factor can satisfy the required value ($F_s \geq 1.2$) for every conceivable circular arc.

An important point to note for construction purposes is that the use of a geotextile does not intend the single-handed reinforcement of an embankment. Particularly careful compacting work is essential to improve the stability of an embankment with friction between the geotextile and well-compacted soil. For this reason, it is desirable for the compaction of the slope section to be conducted by mechanical compaction as in the case of the general section.

Refer to 3.10 – Retaining Walls for the design principles for reinforcement earth walls, the design of which is required to take the earth pressure into consideration because of the fairly steep slope gradient.

(5) Light Weight Embankment Method

Recent years have seen a growing popularity of the light weight embankment method designed to lessen the load on the ground by reducing the weight of embankments. Table 3-3-5 shows the types of light weight banking materials currently in use.

As this method imposes little load on the ground, the decline of slope stability due to an embankment is minimised, making it easy to construct a road on a steep slope. Accordingly, the alteration of nature, which is inevitable with conventional road construction mainly by cutting, can be reduced, hugely contributing to the construction of environment-friendly mountain roads.

Table 3-3-5 Types of Lightweight Banking Materials

Lightweight Banking Material	Unit Volume Weight		Characteristics
	(kN/m^3)	(tf/m^3)	
EPS block	0.1 – 0.3	0.01 – 0.03	Ultra lightweight; synthetic resin foam
Expanded-beads mixed lightweight soil	approx. 7 or higher	approx. 0.7 or higher	Density adjustable; soil-like compaction and deformation characteristics; surplus soil can be used
Air-foam mixed stabilised soil	approx. 5 or higher	approx. 0.5 or higher	Density adjustable; consistency; self-hardening; surplus soil can be used
Coal ash and quenched blast furnace slag, etc.	approx. 10 – 15	approx. 1.0 – 1.5	Granular material; self-hardening
Volcanic ash soil	12 – 15	1.2 – 1.5	Natural material
Hollow structure	approx. 10	approx. 1.0	Corrugated pipe; box culvert; others

(i) EPS Blocks

The standard dimensions for EPS blocks are 2 m x 1 m x 0.5 m (1 m^3) and those of which the unit volume weight is 0.12 – 0.30 kN/m^3 (12 – 30 kgf/m^3), i.e. approximately 1/50 – 1/100 of the soil density, are used for public works purposes.

While the compressive strength is defined by the compressive stress at the time of 5% compression strain in the unconfined compression test, the actual compressive strength of EPS blocks ranges from approximately 70 kN/m^3 (0.7 kgf/cm^2) to 180 kN/m^3 (1.8 kgf/cm^2) depending on the unit volume weight. Self-extinguishable EPS blocks are normally used but the placement near fire, contact with petrol and heavy oil, etc. and long exposure to ultraviolet rays should be avoided. If there is a risk of inundation during or after construction, it is necessary to examine and to prepare measures against buoyancy.

In the case of an embankment on inclined ground in a mountain area, EPS blocks are often used in combination with an earth retaining structure using H-beams, etc. and/or anchor works as shown in Fig. 3-3-10.

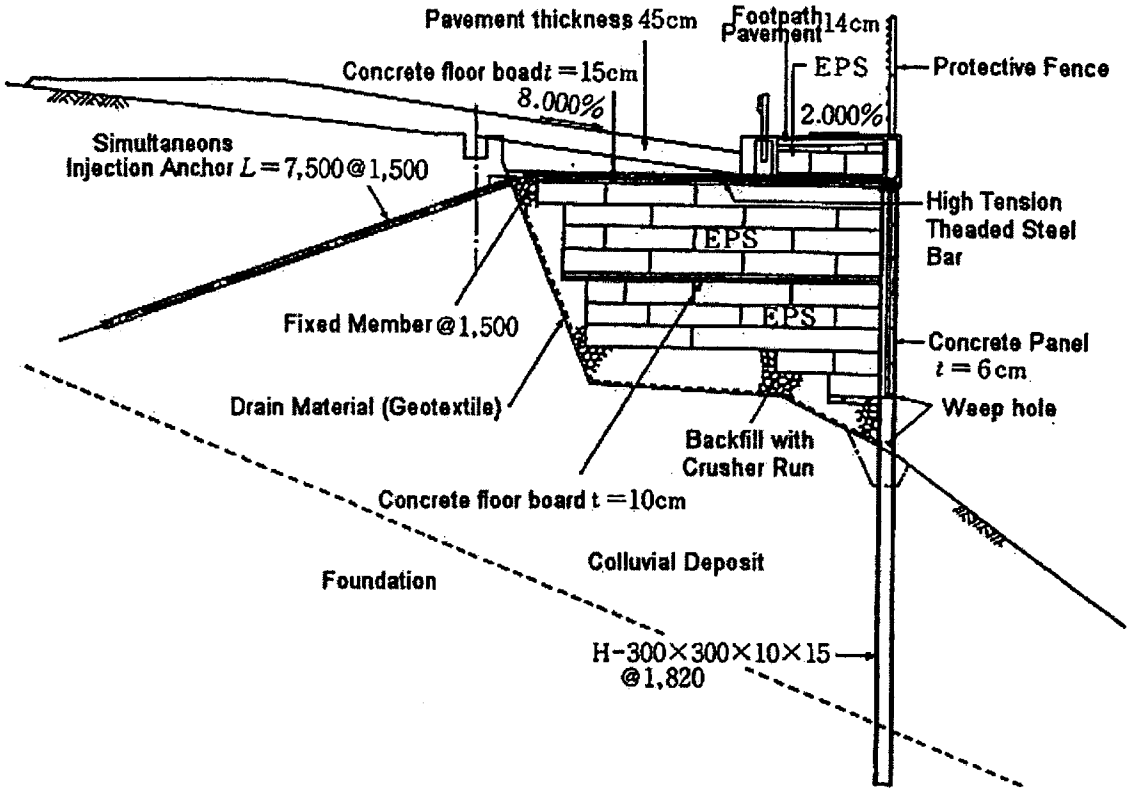


Fig. 3-3-10 Example of Cross-Section of Embankment using EPS Blocks

(ii) Expanded-Beads Mixed Light Weight Soil Method

Expanded-beads mixed light weight soil is a type of artificial light weight soil where ultra light expanded beads (made of expanded polystyrol, expanded polypropylene or similar) are mixed with soil. When strength is required, a stabiliser, such as cement, is often added.

The main characteristics of this artificial soil are its light weight, flexible setting of the strength to suit a specific purpose, handling like ordinary soil, deformability and water-tightness.

(iii) Air-Foam Mixed Stabilised Soil Method

Air-foam mixed soil is a type of light weight stabilised soil produced by mixing such stabilizer as cement and the air foam to slurry with the admixture of water and soil.

The main characteristics of this artificial soil are listed below.

- Lightweight : The unit volume weight can be adjusted in the range of 4 – 15 kN/m^3 (0.4 – 1.5 tf/m^3)

- Fluidity : High level of fluidity before hardened, making it easy to backfill or fill narrow sections.
- Workability : Pumped high pressure feed is possible, making rolling compaction and compaction unnecessary.
- Strength after hardened : The unconfined compressive strength can be set in the range of q_u = 50 – 1,000 kN/m^2 (0.5 – 10 kgf/cm^2) by adjusting the added volume of the stabilizer
- Low horizontal earth pressure acting on a structure when used as a backfill material for retaining walls or abutments.

As many types of soil, including cohesive soil with high water content, can be used as the raw soil, the surplus soil from construction work can be effectively used.

(iv) Coal Ash and Quenched Blast Furnace Slag

As coal ash particles have a hollow structure, their specific gravity is much smaller than that of ordinary soil. In addition, it has the self-hardening property due to a pozzolanic reaction as well as fluidity.

Quenched slag is produced by the rapid water cooling treatment of slag out-pouring from a blast furnace. The particles are glassy and contain air bubbles, indicating a potential self-hardening property. In addition, it enjoys permeability and a certain level of sheer strength because of its compactability and angular particles.

(v) Volcanic Ash Soil

This naturally light weight soil can be used as it is.

(vi) Hollow Structures

Hollow structures, such as corrugated pipes and box culverts, are designed to reduce the weight of an entire embankment.

3.3.6 Embankments Requiring Special Precautions

(1) Embankments on Inclined Ground

In regard to embankments on inclined ground, embankments filling a valley, half-bank and half-cut and boundary of cut and embankments, spring water from the ground often infiltrate the embankment, destabilising the embankment slopes. In such cases, the introduction of a drainage layers inside the

embankment is recommended to reduce the water pressure in the embankment in addition to the introduction of sub-surface drainage to prevent the infiltration of groundwater to the embankment (see Fig. 3-3-11).

As some inclined ground has a soft layer(s) which makes an embankment vulnerable to failure, it may be necessary to conduct investigations on the foundation ground and to design an embankment which is similar to those described in 2.7 – Survey on Landslide.

The shear strength of soil near the ground surface is often low because of weathering and other reasons and, therefore, it is preferable for the ground to be excavated in the form of bench cutting as deep as possible prior to the construction of an embankment. The minimum width and minimum height of bench cutting should be 1 m and 0.5 m respectively.

Lowering of the groundwater level inside an embankment is effective to not only reduce rain-induced disasters but also as an earthquake-proof measure.

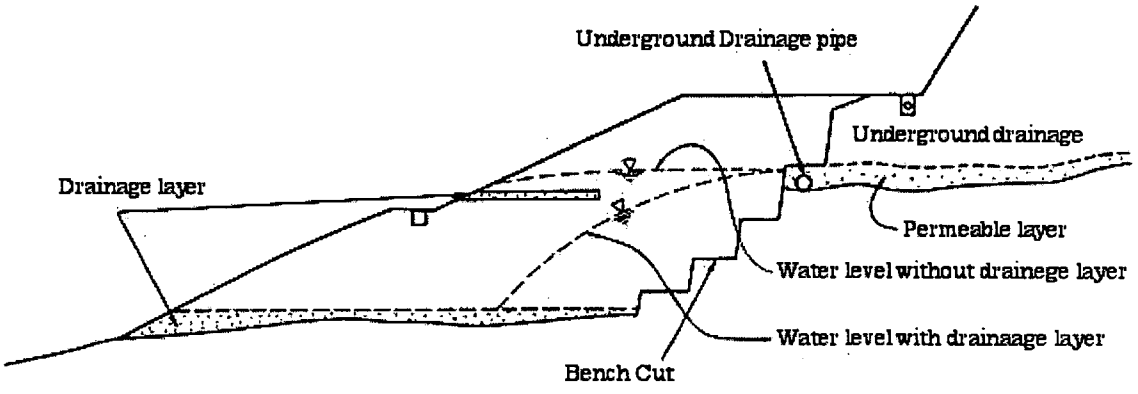


Fig. 3-3-11 Groundwater Drainage Facilities and Drainage Layer for the Embankment on Inclined Ground

(2) Widening of Embankments

When the widening of an existing road is planned by constructing an additional embankment(s) at the side(s) of the road, the construction of the new embankment(s) by the bench cutting of the existing embankment slope(s) as shown in Fig. 3-3-12-(a) is preferable.

There are many examples of embankment failure due to rain or an earthquake at those sites where a narrow embankment has been added to inclined ground. When the slope stability of the ground is secured, much cutting volume should be introduced as shown in Fig. 3-3-12-(b). If possible, the bottom parts of these cut slopes should be made into a horizontal surface above which an embankment is constructed. The widening of an embankment with long-term stability can be successfully achieved in this manner.

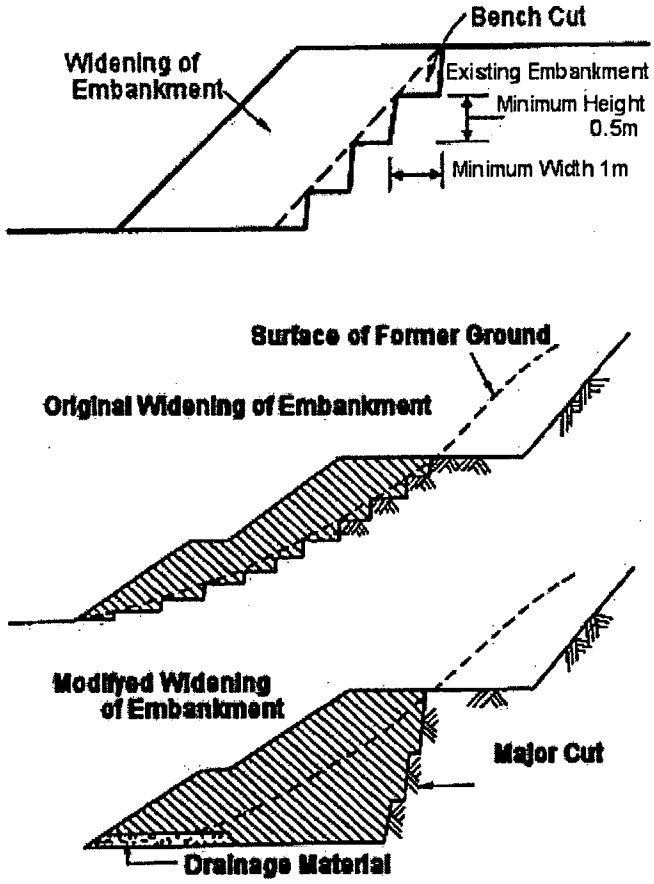


Fig. 3-3-12 Widening of Embankment

(3) Embankments on Soft Ground

Embankments on soft ground face the problems of stability and settlement. Refer to 3.6 – Countermeasures for Soft Ground for further details regarding these problems.

The design and construction of drainage ditches must take their possible settlement into consideration. The locations of vertical drainage ditches in particular should be determined, focusing on places where settlement is likely to be the largest.

(4) Settlement due to Compression of Banking Materials

The muck from tunnel excavation and rocks from a cutting section of a slope consisting of soft rock may be used as banking materials for the lower part of an embankment. In such cases, water infiltrating the voids among the rocks used as a banking material could swell and soften the rocks depending on their properties and compressive settlement due to the weight of the embankment may continue after the completion of construction work. Special attention is required for mudstone which is liable to slaking.

When an embankment is constructed using volcanic clay with a high water content or soil with high clay or silt content, the failure of embankment could occur by the high pore water pressure generated inside the embankment during construction, demanding careful attention. In such cases, slow construction or the introduction of a drainage layer (using a coarse grained material or geotextile, etc.) is necessary.

When cohesive soil with high water content is used as a banking material, rutting by transportation machinery tends to occur. In addition, the strength may decline due to kneading while the embankment itself is compressed due to its own weight. If it takes a long time for this compression to complete, the road surface will experience continued settlement after paving, losing its flatness or even resulting in destruction of the pavement. To combat such a prospect, there is an example of introducing drainage layers at a height of every 5 – 7 m as shown in Fig. 3-3-13 for the purposes of shortening the time for the completion of compression and enhancing the stability of the embankment at the same time.

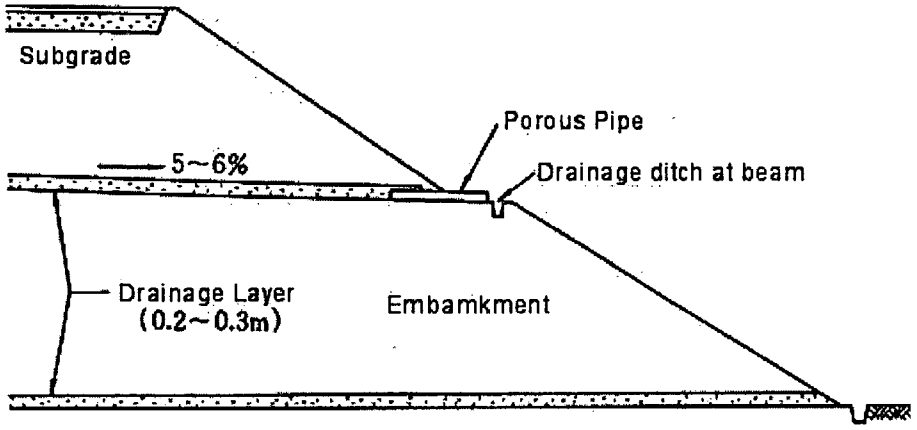


Fig. 3-3-13 Example of Horizontal Drainage Layer inside Embankment

(5) Embankments with Sandy Soil Liable to Erosion by Rainwater

As this type of soil is characterised by poor water retention as well as poor nutrition, protection of the slope surface with top soil, etc. as part of planting is necessary. Meanwhile, facilities with a sufficient drainage function are required to prevent slope erosion by rainwater during construction. For general embankment work, the introduction of a higher cross grade at the central part during construction to create a downward inclination towards the top of the slope and the introduction of a drainage ditch at the top of the slope is recommended.

(6) Embankments at Approach Sections of Structures

The connecting section between such structures as an abutment or culvert and an embankment tends to produce faulting due to differential settlement, resulting in loss of the pavement flatness.

In general, the occurrence of faulting between a structure which is uncompressible and an embankment with a certain degree of compressibility is unavoidable. Faulting is likely to occur at the approach to a structure which is constructed on soft ground but of which the settlement is restrained by the use of bearing piles and others and also at a backfill section for which sufficient compaction using large machinery is difficult. For these sections, careful attention must be paid to the selection of good quality materials and the execution of sufficient compaction (see Table 3-3-6).

Table 3-3-6 Particle Size and Property of Suitable Material for Back-fill

Items	Range
Maximum grain size	Less than 100mm
4.760 μ screen pass percentage by weight	25 to 100%
74 μ screen pass percentage by weight	0 to 25%
Plasticity index (for 420 μ screen pass grain)	Less than 10

Backfilling materials for the approach section of a structure must have such properties as good permeability, ease of compaction, little decline of the strength due to water infiltration and low compressibility. Compaction must be conducted to a higher degree than that for ordinary embankments.

Even if proper precautions are taken to reduce the amount of compression of an embankment at an approach section, the foundation ground may still be liable to settlement. In this case, additional care, such as the employment of the pre-loading method, is necessary to facilitate the settlement of the foundation ground at this section in advance so that there is no prospect of harmful settlement of the road after paving. One preferable example of an approach section of a structure is shown in Fig. 3-3-14.

Other methods to prevent harmful settlement are also available, one of which is the use of an approach cushion made of reinforced concrete plates. These plates are introduced at the banking section of the approach to eliminate sudden and substantial faulting. This approach cushion method is also used for bridges where PC or RC slabs are erected over the side spans of a bridge while adopting so-called earth pressure-relieved spread foundations for the abutments. When a culvert or other structures are introduced across an embankment, they may be allowed to settle to fit the surrounding embankment instead of being supported by non-settling pile foundations or others. In this case, it is necessary to predict the settlement volume due to the embankment so that a camber which corresponds to the predicted settlement volume can be introduced to the

culvert in advance. Fig. 3-3-15 shows examples of reducing the impediments at the approach section of a structure by means of careful design.

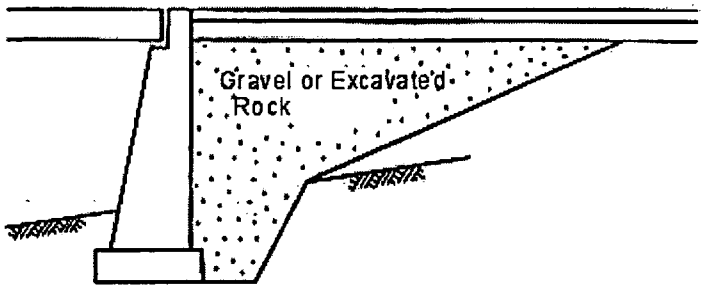


Fig. 3-3-14 Example of Treatment at Approach Section to Structure

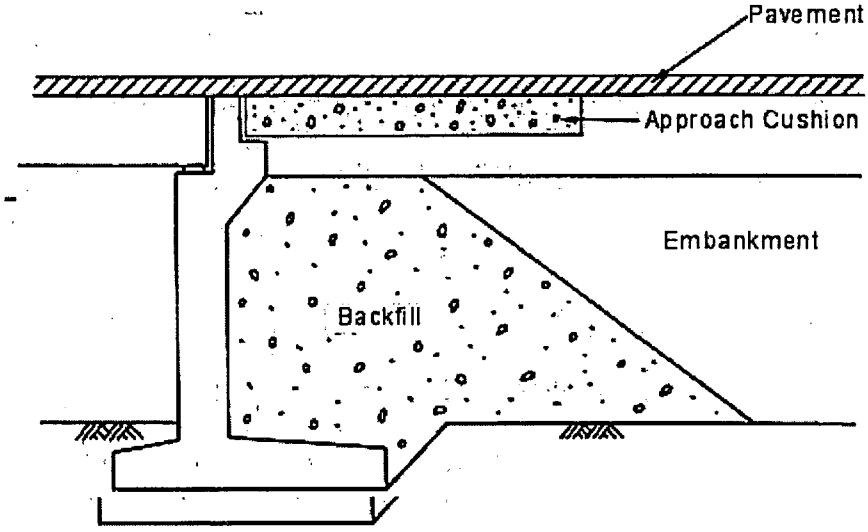


Fig. 3-3-15 Idea to Reduce Faulting at Approach Section to Structure

(7) Embankments Liable to Earthquake Damage

In the past, there have been many cases of embankments being considerably damaged by earthquakes due to liquefaction of the loose saturated sandy ground on which the embankments are constructed. When exposed to vibration, such loose saturated sandy ground significantly loses its strength due to an increase of the excess pore water pressure. In contrast, the strength of clayey ground does not decline much at the time of an earthquake except for that with a high sensitivity ratio. However, an embankment on soft clayey ground with essentially weak strength may be liable to severe damage by an earthquake. Even if the ground is not soft, major embankment failure occurs when exposed to seismic force under the condition of a high water content of the embankment body.

The approach section of a bridge or embankment at both ends of a crossing structure, such as a culvert, may experience faulting due to settlement at the time of an earthquake. Effective measures to prevent faulting

include thorough compaction using high quality banking materials and the introduction of an approach cushion.

In the case of a road employing half-bank and half-cut on inclined ground, the banking section often experiences settlement because of faulting at the boundary between the cutting section and the banking section.

Earthquake-proof measures for embankments include the improvement of soft ground, thorough compaction of the embankment and the installation of drainage facilities to reduce seepage water to the embankment. Prevention of the liquefaction of the embankment itself at the time of an earthquake requires the selection of materials of which the particle sizes are resistant to liquefaction, thorough compaction and good drainage. The use of the reinforced earth embankment method using a geotextile or the light weight embankment method is effective to prevent sliding failure and/or the differential settlement of an embankment.

3.4 Slope Protection

3.4.1 Selection of Slope Protection Works

Slope protection works aim at preventing the erosion and weathering of a slope by covering a slope with vegetation or a structure or stabilising a slope with drainage works or a retaining structure. The standard types of slope protection works are shown in Table 3-4-1 and these can principally be classified into those involving vegetation and those involving a structure.

For the selection of suitable slope protection works, it is essential to consider such geological and soil conditions of the slope as the rock and soil properties and pH value, situation of spring water and water catchment, weather conditions and size and gradient of the slope together with such aspects as construction cost, working conditions, maintenance and conservation of the landscape/environment.

Vegetation works are performed to prevent erosion due to rain water by growing plants on the faces of slope and by firmly binding the faces with roots of plants and to ease the temperature change on the ground surface and to provide beautiful views created by greening, to control frost heave collapses. Vegetation is frequently used in places where the vegetation is possible and since the costs for vegetation are relatively low in most cases.

Table 3-4-1 Main Types of Slope Protection Works and Their Purposes

Category	Type of Work	Purpose and Characteristics
Vegetation	▪ Seed spraying ▪ Borrow spraying ▪ Planting ground spraying ▪ Sodding ▪ Sodding mat ▪ Planting sheet	Erosion prevention; control of collapse due to frost heaving; total vegetation cover
	▪ Simple seed matting work ▪ Simple sodding work	Erosion prevention and partial vegetation of embankment slope
	▪ Planting sandbags	Erosion prevention of slope surface made of poor soil or hard soil
	▪ Spraying with planting of young trees	Erosion control and landscaping
	▪ Planting	Landscaping
Slope Protection by Structure	▪ Net hurdling ▪ Gabion works	Control of sediment outflow at slope surface due to erosion or spring water
	▪ Precast concrete cribworks	Erosion prevention when filled up with soil or cobblestones
	▪ Mortar/concrete spraying ▪ Stone pitching ▪ Block pitching	Prevention of weathering, erosion and seepage of surface water
	▪ Concrete pitching ▪ Spray cribworks ▪ Cast-in-place concrete cribworks	Prevention of collapse of slope surface; retaining at sites likely to face slight earth pressure; prevention of exfoliation of rock base
	▪ Stone/block masonry retaining wall ▪ Gabion works ▪ Crib retaining wall ▪ Concrete retaining wall	Resistance to a certain level of earth pressure
	▪ Reinforced earth (reinforced earth embankment, reinforced cut slope method) ▪ Rock bolt works ▪ Ground anchor works ▪ Pile works	Resistance to sliding force of sliding soil mass

However, as the root system of such planting work remains relatively shallow, planting is not very effective for slopes liable to deep sliding. Moreover, plants find it difficult to grow under such structures as viaducts and bridges with little sunlight or rainwater or with a rock base with little soil or strongly acidic soil. Even in the case of slopes of which the gradient is approximately 1:0.8 to 1:1.2, planting alone may not be sufficient to prevent surface erosion or failure depending on the soil properties. For these slopes, it is necessary to apply slope protection works with a structure.

There are many types of slope protection works with a structure, ranging from relatively small-scale works aimed at stabilising an embankment for effective planting to relatively large-scale works aimed at preventing the weathering, erosion or surface failure of a slope and further to very large-scale works designed to prevent slope failure. Among the various types of slope protection works, while retaining wall works, pile works, cast-in-place concrete cribworks complemented by anchor works and reinforced earthworks can be assumed to have a certain

degree of resistance to earth pressure, other types of slope protection works are not essentially designed for use in unstable places subject to earth pressure. Accordingly, the application of different measures is necessary if these works are subject to earth pressure due to a change of circumstances in the future.

If a slope experiences spring water, the employment of slope drainage works in addition to slope protection works is essential to prevent the scouring as well as destabilisation of the slope.

As the soil properties or the state of spring water are often not uniform even at a single slope, the types of works which are suitable for the specific conditions of different places must be selected. In this case, the primary principle is to introduce heavy works at the lower part of the slope and lighter works at the upper part of the slope. The selection of similar works is preferable as the use of different types of works to protect a small area each is detrimental to the landscape.

The on-site working conditions are often very complicated and the purposes of slope protection works tend to overlap with each other. Suitable slope protection works should, therefore, be carefully selected, taking the likely cost into consideration. The standard flow for the selection of slope protection works is shown in Fig. 3-4-1 and Fig. 3-4-2 for reference purposes. The following points should be referred to as judgement criteria when individual judgements are required in this flow.

*1 As a stable gradient corresponding to the soil properties of the ground, the average gradient of the standard slope gradients for different soil properties of the ground shown in Table 3-2-1 should be used as a yardstick. In the case where such a stable gradient cannot be secured, re-cutting should be conducted if possible.

*2 The existence of a rockfall hazard should be judged with reference to 2.6 – Survey on Rockfall and 3.7 – Countermeasure for Rockfall.

*3 Classification of the ground follows the relevant guidelines described in 2.10 – Soil Investigation.

*4 Mudstone and shale of the Tertiary and tuff and serpentinite with low solid state are liable to weathering due to the release of stress by cutting, subsequent repetition of drying and wetting and repeated action of freezing and melting.

*5 The average gradient of the standard slope gradients for uncondensed soil is used as the yardstick for a stable gradient which does not lead to slope failure despite the progress of weathering.

*6 Strata mainly consisting of sandy soil, such as the pit sand and the terrace gravel stratum are particularly vulnerable to erosion by surface water.

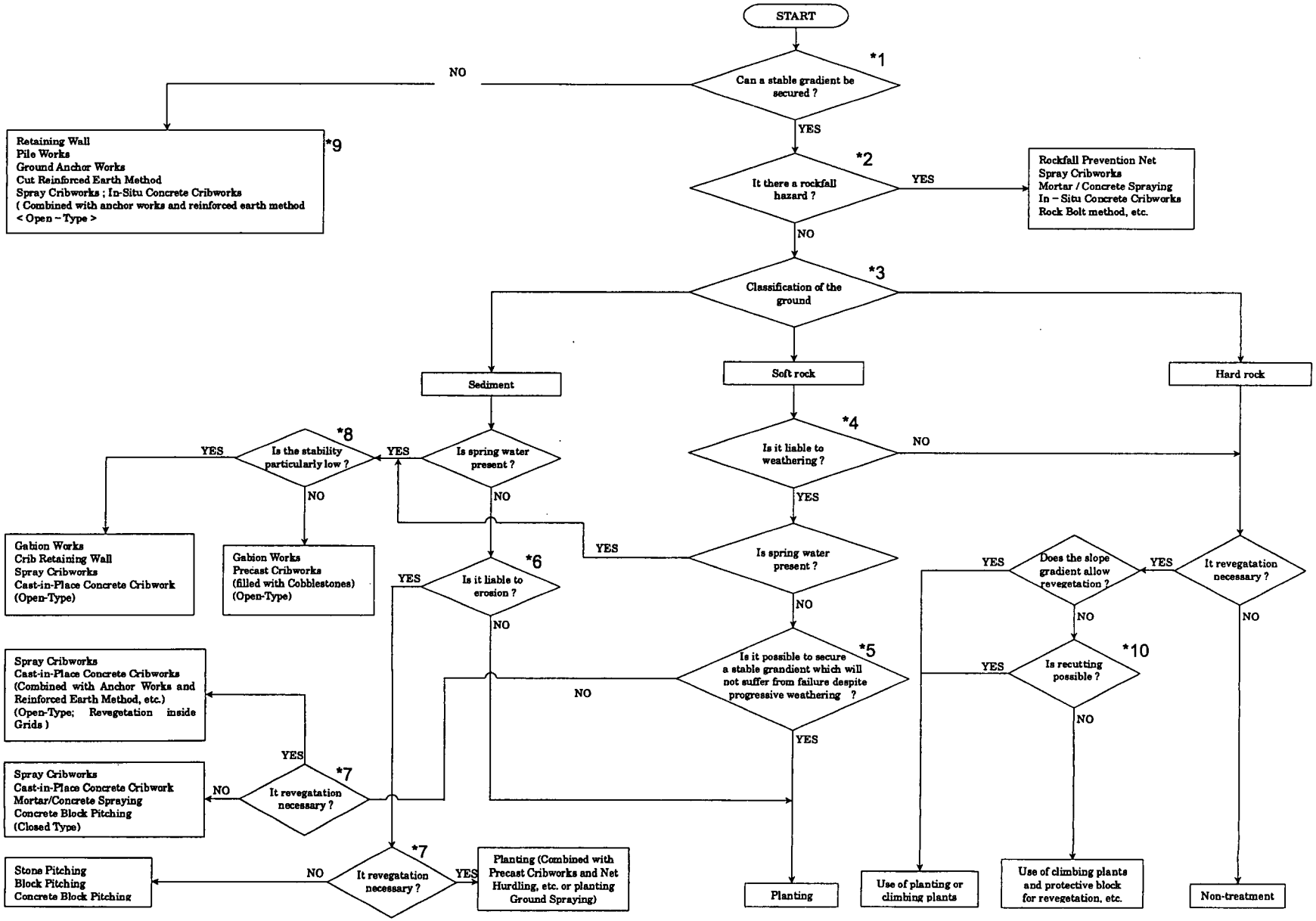


Fig. 3-4-1 Selection Flow of Slope Protection Works for Cut Slope

*7 The suitable type of slope protection works should be judged taking mitigation of the impacts on the natural environment, harmony with the surrounding landscape and continuity of the target vegetation, etc. into consideration.

*8 The suitable type of slope protection works should mainly be judged based on the expected degree of stability, and mat gabions, crib retaining walls, shotcrete cribworks or cast-in-place concrete cribworks should be applied when the perceived degree of stability is particularly low.

*9 In the case of a slope where slope protection works with a structure are conducted, planting is additionally conducted if such work is required as an environmental or landscape preservation measure.

*10 Re-cutting here means re-cutting for revegetation purposes.

*11 The average gradient of the standard slope gradients for banking materials and banking height shown in Table 3-3-2 is used as the yardstick to determine the stable gradient of an embankment slope.

*12 Rock crushed muck here mainly means hard debris which is unlikely to become fragile due to weathering. Other types of muck are considered to be similar to general soil.

*13 Typical banking materials liable to erosion are sand and sandy soil.

*14 Slope protection works which are resistant to erosion by rainwater, etc. should be selected.

Several types of slope protection works are available for different types of failures and some examples are shown in Table 3-4-2.

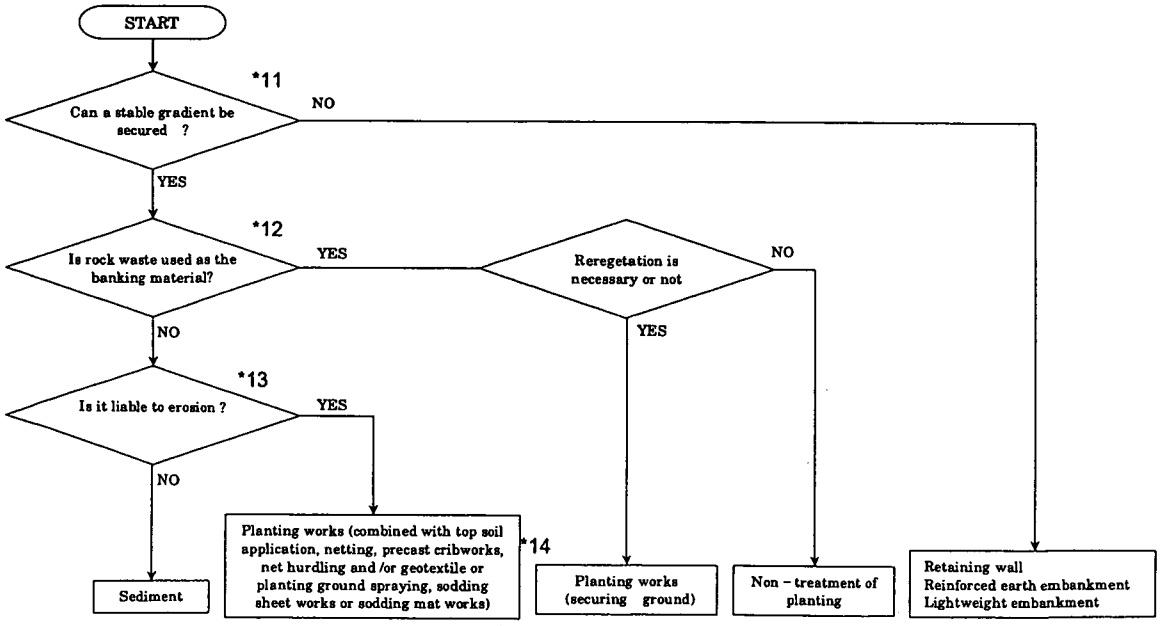
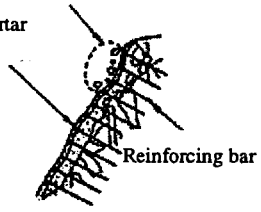
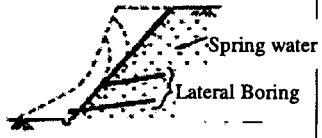
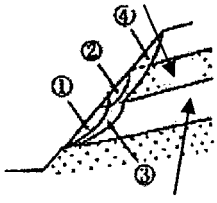
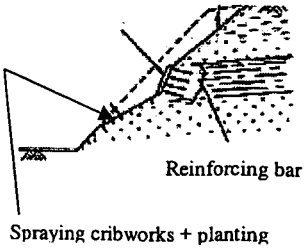
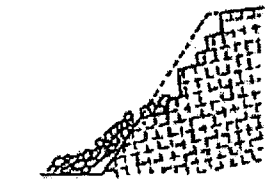
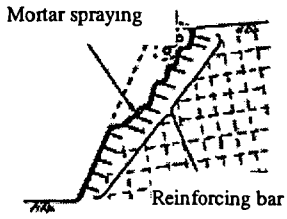
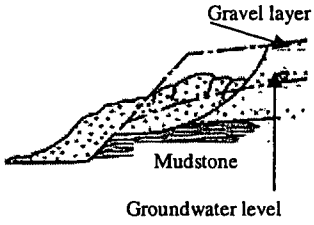
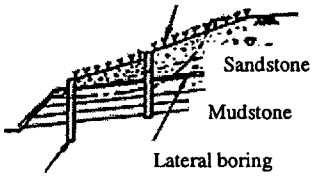
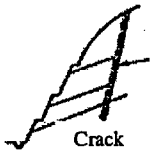
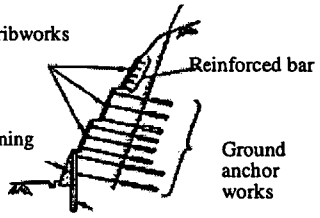
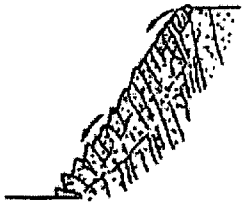
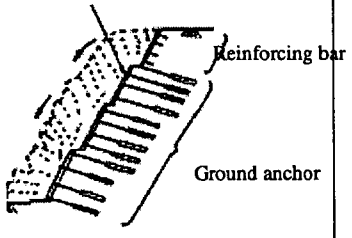


Fig. 3-4-2 Selection Flow of Slope Protection Works for Embankment Slopes

Table 3-4-2 Type of Failure of Cut and Natural Slopes and Examples of Slope Protection

Category	Description	Type of Failure	Example of Slope Protection
Erosion/Fall	1) Separation of the surface or production of a gully by repeated drying and wetting, freezing and rain; could move to a deep failure if unarrested.	 Gully erosion by surface water	Precast concrete cribwork  Netting + planting or precast concrete cribwork + planting
	2) Fall of an overhang section on a slope		Planting  Recutting + planting
	3) Fall of rocks with plenty of cracks and joints	 Loose stone type rock fall	Removal of loose stones Mortar  Reinforcing bar Mortar spraying (removal of loose stones) + reinforced earthworks
Surface Failure	1) Slipping of the lower heavily weathered rock layer due to the slipping of top soil, often induced by spring water	 Spring water Failure caused by piping	Precast concrete cribwork  Spring water Lateral Boring Recutting + precast concrete works (filled with cobblestones) + lateral boring
	2) Slipping of the surface layer as a result of weathering, etc.	Sandstone  Mudstone Surface failure with progress of weathering, etc.	Spraying Mortar Spraying  Reinforcing bar Spraying cribworks + planting Cutting with different gradients + spraying cribworks + reinforced earthworks + planting + mortar spraying

	3) Slipping of rocks along a dip slope structure or cracks in the bedrock (joints, small fault and/or thin layer); frequently wedge-shaped failure in the case of the latter	 Failure along cracks of rock	Removal of unstable rocks  Mortar spraying + reinforced bar
Large-Scale Failure/ Landslide Type Failure	1) Large scale slipping of a slope consisting of a soft layer with a low degree of consolidation or a slope with unstable factors in terms of the geological structure following a rise of the groundwater level	 Slipping along surface of discontinuity in terms of permeability	Planting  Pile works Recutting + pile works/lateral boring + planting
	2) Large-scale slipping of a rock body which has such a geological structure as a dip slope, fault or fracture zone	 Sliding along fault fracture zone	Spray cribworks  Pile works + spray cribworks + ground anchor works + reinforced earthworks + concrete lining
	3) Overturning of a dip slope or rocky slope with developed cracks	 Overturning of dip slope	Spray cribwork  Spray cribworks + reinforced earthworks + ground anchor works

3.4.2 Important Points for Introduction of Slope Protection Works

(1) Slope Gradient Suitable for Plant Growth

Although it depends on the form of plant colonies to be created and the method of plant introduction, it is generally safe to assume that planting alone should be sufficient to prevent the erosion or surface failure of slopes to a certain extent provided that the slope gradient is gentler than 1:1.0 – 1:1.2 for soft rock and cohesive soil or 1:1.5 for sand and sandy soil. When the slope gradient becomes steeper than that above, it becomes difficult to maintain the slope stability with planting alone, necessitating the concurrent use of cribworks or net hurdling (see Table 2-2-3).

(2) Slopes Consisting of Sandy and Other Soil Liable to Erosion

When cut slopes are made of sandy and other soil liable to erosion with little spring water, planting alone is generally employed. However, if prevention of erosion by surface water is required, cribworks or net hurdling is employed in addition to planting. Where there is much spring water, gabion works, cribworks filled with cobblestones or net hurdling, etc. is employed depending on the actual level of spring water. The installation of sub-surface drainage works in a tree-like manner with further protection by concrete blocks, etc. covering such works is highly effective to prevent scouring at the back of slope protection works. The introduction of drainage facilities at slope shoulders and each berm regardless of the amount of spring water is also desirable.

Embankment slopes consisting of sandy soil should preferably be protected by top soil of some 30- 50 cm or more in thickness. In the case of a high embankment, the toe of slope is vulnerable to scouring or failure due to seepage water, resulting in mudflow. In such an area, the introduction of a drainage layer or sub-surface drainage works in addition to planting or the employment of a combination of net hurdling, pre-cast cribworks and/or a block masonry retaining wall with planting is necessary.

(3) Slopes with Much Spring Water

In regard to slopes with much spring water, such underground drainage facilities as sub-surface drainage works and horizontal drainage holes should be actively introduced together with the application of such open-type slope protection works as crib retaining walls, gabion works and cribworks filled with cobblestones as slope protection works.

(4) Rock Slopes with Small-Scale Rockfall Hazard

Among slopes with a rockfall hazard, those made of gravelly soil or weathered soft rock is liable to small-scale rockfall. For these slopes, the installation of a rockfall prevention net to hold down loose stones and/or rockfall prevention fences to prevent rocks from falling on to the road surface along with planting is necessary. In the case of highly cracked soft rock without spring water, mortar concrete spraying is appropriate. It is preferable to use rockfall prevention works for slopes containing an exfoliation-type rockfall on highly cracked slopes of hard rock and the additional use of rockfall protection works is even more desirable if the gradient of these slopes is very steep.

(5) Slopes Composed of Hard Soil

It is required to perform digging and soil dressing at some portions of a slope so as to allow the roots to grow, and sodding should then be performed on the dressed soil of hard slopes made of dense sandy soils (exceeding the soil hardness of 27 mm), hard clayey soils (exceeding the soil hardness of 23mm), soft rocks or hard clay.

(6) Slopes Made of Soil with Problematic Acidity

Plant growth is difficult when the pH value of soil is lower than 4 from the beginning or where an old stratum of raised sludge from the bottom of a lake is liable to the quick change to extremely acidic soil due to sudden exposure to air due to cutting. At such sites, it is desirable to replace the original soil with borrowed materials, to neutralise the soil with lime, to introduce drainage or waterproofing measures which are appropriate for the specific site conditions in order to prevent any adverse impacts by seepage water which is highly acidic because of the properties of the parent rock making up the slope on the growth of introduced plants or to employ slope protection works with such a structure as pitching work.

(7) Slopes with Non-Uniform Conditions of Soil Properties and Spring Water

As the soil properties and the state of spring water are often not uniform even at a single slope, the types of works which are suitable for the specific conditions of different places must be selected. The selection of similar works is preferable together with treatment of the ground with drainage and other necessary works as the use of different types of works to protect a small area each is detrimental to the landscape.

3.4.3 Planting

(1) Characteristics of Planting

Planting is conducted with the expectation that, once fully grown, plants can perform such functions as preventing slope erosion and the blockage of rainwater, mitigating the strong impact of raindrops on the ground, reducing the velocity of the surface flow, holding of soil by the root system and increasing the seepage capacity of the ground surface. In addition, harmonization with environment and landscape in surrounding area can be expected. As the growth depth of plant root system is limited, however, planting cannot be expected to prevent the deep failure of slopes. The following preconditions are attached to planting because of the use of live plants as the material.

① State of ground: The ground for plant growth must be stable vis-à-vis erosion or failure.

② Plant selection: The selected plants must be compatible with the geological properties and gradient of the subject slope and the local weather conditions and must also be compatible with the purpose of vegetation work.

③ Work method: The work method must ensure the continual deterrence of erosion until full plant growth is achieved so that the plants can permanently provide the required slope protection.

④ Work timing: Planting must be conducted during a period where a suitable temperature, moisture content and sunlight, etc. are ensured until such a time when the germinated plants grow sufficiently to resist erosion.

⑤ Extraordinary weather, diseases and pests: There must be no unfavourable external factors for plant growth.

When the introduction of planting is required for slopes which fail to meet the above conditions, such as slopes with an unstable surface or slopes where planting is difficult, combination with a structure, the creation of new bed for growth of plants or the adoption of an advanced plant management technique should be considered.

(2) Establishment of Planting Targets

While the primary purpose of planting is the prevention of erosion as well as surface failure, it is important at the same time to carefully select suitable plants with a view to harmonising with the surrounding environment and reducing the maintenance work volume, taking the full growth situation of the plants in the future into consideration. When establishing the planting targets, the relevant policies must take maintenance after initial planting into consideration.

There are three types of target plant colonies for vegetation based on natural appearance, i.e. high forest type, shrub type and grassland type. In addition, there is a special type for landscaping. For planting design, vegetation targets should be established taking the surrounding landscape and the maintenance conditions into consideration and types of plants and a work method corresponding to the slope surface conditions should be examined.

In general, in the case of mountain areas and areas where the natural landscape is deemed to be important, it is preferable to opt for a high forest or vegetation to shift from shrub to a high forest from the viewpoints of establishing continuity with the surrounding vegetation and reducing the maintenance work burden. In suburban areas and areas near farmland or pasture, it may sometimes be preferable to maintain a good view by means of planting herbaceous plants and/or flowering trees even though these require more maintenance work (Table 3-4-3).

Table 3-4-3 Types of Target Plant Colonies

Target Type of Plant	Target Appearance	Conditions of Applicable Sites
High Forest Type	Community dominated by trees	▪ Surrounded by forest ▪ Inside a natural park
Shrub Type	Community dominated by shrubs	▪ Surrounded by forest ▪ Surrounded by farmland
Grassland Type	Community dominated by herbaceous plants	▪ Surrounded by grassland ▪ Surrounded by farmland ▪ Mortar sprayed plane
Special Type	Special or artificial plant colony	▪ Sites where special consideration is required in regard to the surrounding landscape or natural environment

(3) Types and Characteristics of Planting and Bed for Plants

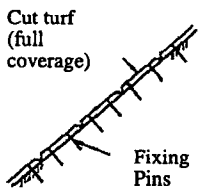
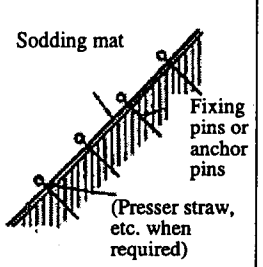
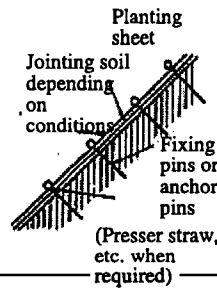
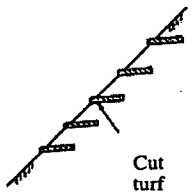
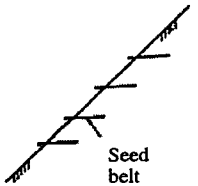
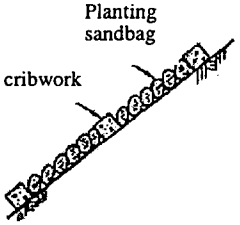
There are many planting methods which are suitable for different species as well as topographical, geological and meteorological conditions. As the selection of the incorrect method results in failure to achieve the targets and purposes of planting, careful examination is necessary in regard to the plants to be used and the method of introducing such plants. Basically, the selected method should be able to firmly fix the bed for plants which satisfies the germination and growth requirements of the target plants to the slope. The nature and thickness of this bed are determined by the degree of the plants' demand for fertiliser as well as the germination characteristics and the soil properties and gradient of the target slope.

Table 3-4-4 shows the characteristics of various types of artificial planting. Meanwhile, Table 3-4-5 shows the types, characteristics and points to note for the application of bed for growth of plants which is to be conducted together with planting.

Table 3-4-5 Characteristics of Works of Bed for Plant and Important Points for Application

Type		Characteristics	Important Points
Drainage		▪ Prevention of sliding surface failure due to increased groundwater or erosion due to the running down surface water ▪ Improvement of air permeability ▪ Removal of acidic water	▪ Assured water collection and structure to prevent seepage to the slope surface ▪ Cross-sectional area eliminating overflow and non-leaking structure for a drainage ditch; assured waste water disposal at the end of drainage
Cribworks	Spray Cribworks Cast-in-Place Concrete Cribworks	▪ Capable of creating a structure to deal with different shapes and sizes of failure at the shallow end of a slope ▪ Possible application of planting inside cribworks ▪ Spray cribworks are capable of dealing with undulations of the slope surface	▪ Careful examination is required when applied to a slope consisting of expansive or contractive rock or soil of which the freezing depth could be deep
	Precast Concrete Cribworks	▪ Capable of fixing and maintaining soil or planting sandbags as revegetation foundations to the slope surface	▪ Must be avoided when bulging or frost heaving is expected to occur as this type of works cannot resist the soil pressure of the slope surface ▪ Applied to slopes of which the gradient is gentler than 1:1.0 and where the cribworks will not sink due to scouring, etc.
Net Hurdling		▪ Partial fixing of slipping sediment, mitigation of the speed of descending water or cushioning of falling rocks or avalanche	▪ In principle, should be used in combination with planting ▪ The introduction of woody plants (seeding or planting of young trees) to secure proper functioning in the future together with net hurdling is desirable
Netting	Metal Netting	▪ Effective to prevent exfoliation of the slope surface due to the running down of surface water or frost heaving, etc., to maintain artificial foundations and to prevent rockfall	▪ A net with too fine a mesh or a long-life net may be detrimental to the growth of woody plants
	Textile Netting	▪ Effective to prevent exfoliation of the slope surface due to the running down of surface water and to maintain artificial foundations	▪ Difficult to deal with frost heaving or rockfall due to the lack of rigidity
Windbreak Works		▪ Netting with a fine mesh and fencing works are useful to alleviate the drying of plumules and seedlings and wind damage	▪ Careful analysis of the wind direction, wind force and expected degree and scope of the effects is required
Planting Sandbags		▪ Useful to secure a rooting area on the slope surface and to maintain fixed roots	▪ Careful examination of the required mesh and durability of the sandbags is required ▪ Applicable to a gentle slope surface of which the gradient is less than 1:0.8

Table 3-4-4 Types and Characteristics of Artificial Planting

Type		Sodding	Sodding Mats	Planting Sheet	Simple Sodding	Simple Seed Matting	Planting Sandbags
Work Method		Sodding over the entire area	Sodding over the entire area	Sodding with full coverage or in strips	Placing of cut turf while applying top soil	Created of seeded belts while applying top soil	Fixing of planting sandbags
Materials to be Used	Foundation Material	Cut turf; rolled turf	Mat-like material with some thickness containing seeds, fertiliser and foundation material for growth	Sheet-like material with attached seeds and fertiliser	Cut turf	Textile belts with attached seeds and fertiliser	Textile bags willed with soil/improved soil and seeds, etc.
	Plants	Cut turf field turf Rolled turf exotic herbaceous plants	Seeds of exotic or native herbaceous plants	Seeds of exotic or native herbaceous plants	Field turf	Seeds of exotic or native herbaceous plants	Woody seeds; seeds of exotic or native herbaceous plants
	Fertiliser	Complex fertiliser Slow-release fertiliser	Advanced complex fertiliser	Advanced complex fertiliser	Complex fertiliser; slow-release fertiliser	Advanced complex fertiliser	Compost; PK complex fertiliser; slow-release fertiliser
Supplementary Materials		Fixing pins; sowing soil; jointing soil	Fixing pins; anchor pins	Fixing pins; anchor pins; sodding soil or jointing soil			Fixing pins; anchor pins
Accompanying Work							Grooving; cribworks
Erosion Resistance After Work		Relatively large	Large	Large	Small	Small	Large
Application Conditions	Geology	Cohesive soil (hardness: <27 mm) Sandy soil (hardness: <23 mm)	As left	As left	As left	As left	Sediment with little nutrients or hard sediment/rock
	Gradient	Gentler than 1:1.0	Gentler than 1:1.0	Gentler than 1:1.0	Gentler than 1:1.2	Gentler than 1: 1.2	Gentler than 1:0.8
Remarks		Applied to a small area where landscaping effect is required	Necessary to ensure close contact of the mats with the slope surface	- Necessary to ensure close contact of the sheet with the slope surface - Additional fertiliser application may be necessary for soil with little nutrients	- Applied to small embankments - Unsuitable for sandy soil	- Applied to small embankments - Additional fertiliser application may be necessary for soil with little nutrients - Unsuitable for sandy soil	- May fall if the gradient is steeper than 1:0.8 - Soil with excellent fertiliser retention should be used if herbaceous seeds are used
Standard Drawing							

3.4.4 Slope Protection Works with Structures

Various types of slope protection works with structures are conducted for slopes which are unstable without slope protection and which are unsuitable for vegetation, slopes of which the long-term stability against erosion cannot be guaranteed by only vegetation or slopes which are liable to failure and/or rockfall. Table 3-4-1 lists the different types of slope protection works with structures which are already described in 3.4.1 – Selection of Slope Protection Works.

(1) Stone and Concrete Block Pitching

Stone pitching and concrete block pitching are mainly used to prevent slopes from being eroded and weathered and are employed for non-cohesive soil, mudstone or easily collapsible clay slope with a slope gradient gentler than 1:1.0.

In addition, they can be used for saving land by making the slope gradient steeper than the standard and for protecting the backfilled portions of overbridges, or earth pressure relief type abutments. Materials for stone pitching and concrete block pitching and thickness of materials are to be determined depending upon the gradient of slope and purpose of use, and are normally as shown in Table 3-4-6.

Table 3-4-6 Slope Gradient and Thickness of Stone and Concrete Block (Unit: cm)

Location / Type of Pitching / Slope Ratio [1]	Protection of Ordinary Slope			Protection at Special Places (Backfilling of Over-Bridges, Earth Pressure Relieved and Front of Abutments, etc.)		
	Stone Pitching	Concrete Block Pitching	Concrete Plate Pitching	Stone Pitching	Concrete Block Pitching	Concrete Plate Pitching
1.0 – 1.2	35 – 25 [1]	35	<20	35	35	<20
1.2 – 1.5	35 – 25 [2]	35	<20	-	25	<20
1.5 – 1.8	<25	<12	<20	-	<18	<20

Notes
1) Applied to slopes of less than 5 m in height when the gradient is steeper than 1:1.5.
2) Boulders are used for the case of thickness of 25 cm for stone pitching to slopes of less than 3 m in height.

The height and slope length are generally not more than 5 m and 7 m respectively.

In the case of stone pitching, it is preferable to adopt as gentle a gradient as possible because of the difficulty of tightly binding the stones.

If spring water or seepage water is present, backfilling with cobblestones or crusher-run is required to ensure good drainage at the back of the pitch works (see Fig. 3-4-3).

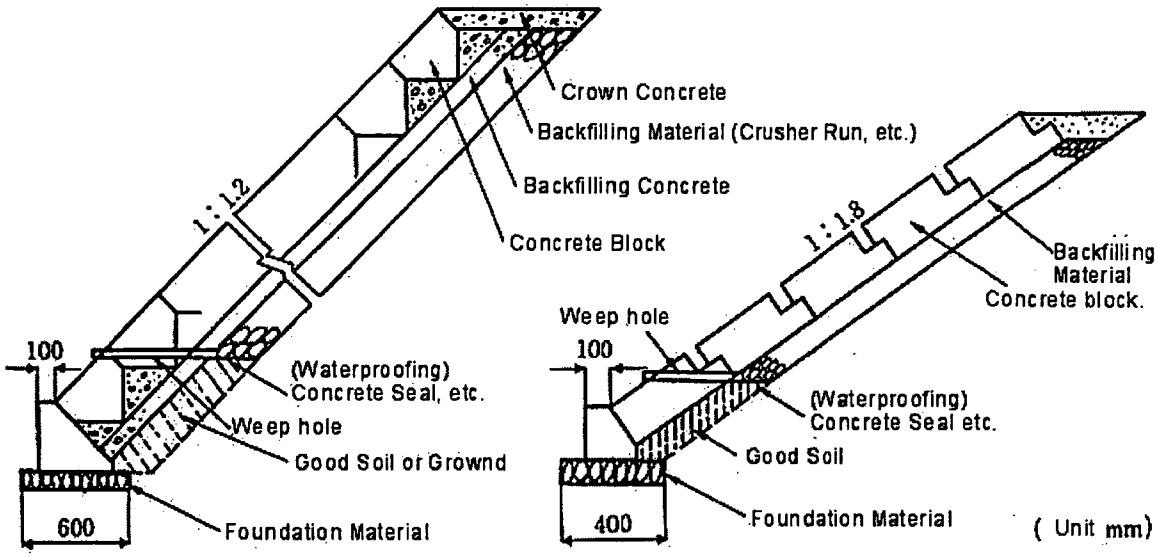


Fig. 3-4-3 Examples of Concrete Block Pitching

Weep holes with a diameter of approximately 50 mm should be introduced at a rate of one every 2 – 4 m^2 as standard. The density of these holes should be increased at sites where the amount of spring water is large. At such sites, stone pitching should be conducted following the installation of sufficient drainage facilities (drainage conduits and waterproofing mats, etc.) in view of the fact that many slopes have collapsed in the past due to the lack of such precautionary measures.

(2) Concrete Pitching

This is used when concrete block cribwork or mortar spraying is considered to be not appropriate for the slope of bedrock with many joints or loose talus cone layer.

For large or steep slopes, it is desirable to reinforce the concrete with reinforcing bars or wire mesh and also to install non-slip anchor pins or anchor bolts (refer to Fig. 3-4-4).

The plain concrete pitching will require a minimum thickness of about 20 cm. In principle, non-slip anchors should be placed at a rate of one anchor per 1 to 2 m^2 , and the depth of embedment should be 1.5 to 2.0 times the thickness of the concrete. It is important to determine the length of anchor pins or anchor bars to suit such specific purposes as the prevention of slope failure and slippage of the concrete pitching depending on the geological conditions of the site.

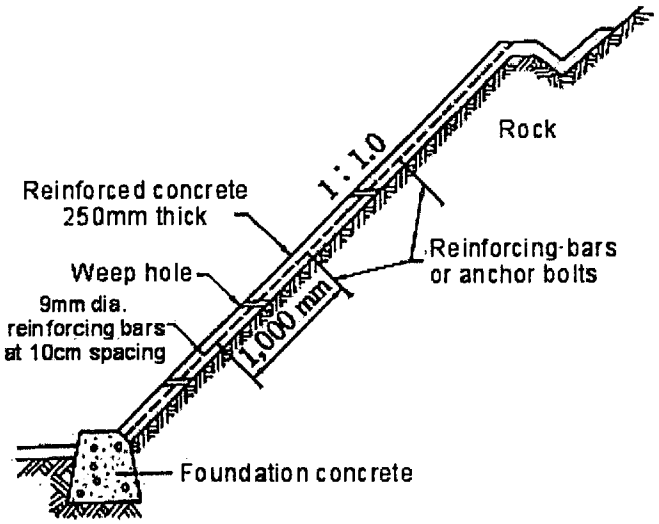


Fig. 3-4-4 Example of Concrete Pitching

(3) Mortar and Concrete Spraying

This is used on easily weatherable rock and rock likely to be weathered and stripped off even though there is no spring water in slope and there is no danger for the time being as well as for mudstone not suited to vegetation.

The thickness of spraying is determined by taking account of the slope conditions and weather conditions, and the standard thickness of spraying is 8 to 10 cm for mortar spraying and 10 to 20 cm for concrete spraying.

Prior to spraying, it is required to lay the wire mesh over the face of slope and to anchor it. The standard number of anchors is 1 to 2 every square meter. When spring water is present on the spraying surface, the introduction of sufficient drainage facilities is necessary (Fig. 3-4-5).

When thick spraying is required because of a steep gradient or when the slope surface is considerably uneven, an increase of the number (density) of anchor pins or anchor bars is desirable depending on the actual need.

As part of spraying, weep holes should, in principle, be introduced at a standard rate of at least one hole per 2 – 4 m^2 of the spraying area. As the durability of spraying is significantly affected by the weather conditions, particularly during the work, in addition to the composition of the admixture, spraying work conditions, equipment performance and skill of workers, careful attention must be paid to the timing of the work and the working hours.

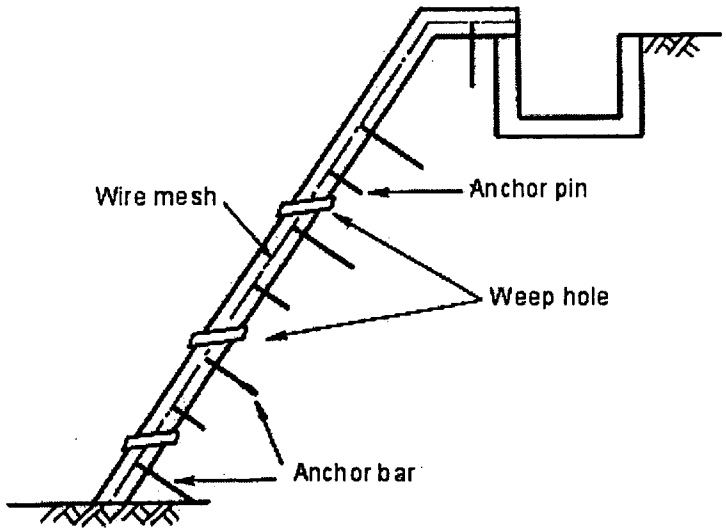


Fig. 3-4-5 Example of Mortar Concrete Spraying

In principle, spraying should not be conducted in the following cases.

① Normal spraying work is severely disturbed by strong wind.
② Proper curing cannot be expected because the temperature is near freezing point.
③ The cement is likely to be washed away from the spraying surface because of strong rain.
④ Spraying operation is likely to be disturbed by extreme drying due to strong wind and fine weather.

While the guidance admixture ratios by weight of cement, sand and aggregate are C:S = 1:4 (C: cement weight; S: sand weight) for mortar spraying and C:S:G = 1.4:1 – 1:4:2 (G: aggregate weight), a cement volume of more than 400 kg/m^3 and the adoption of a water-cement ratio of not more than 60% are desirable. The strength of the mortar and concrete should be decided after trial mixing using 15 N/mm^2 (150 kgf/cm^2) as the yardstick.

(4) Precast Cribworks

Precast cribworks is employed at cut slopes liable to erosion, sites where revegetation is unsuitable because of the specific conditions despite the slope having a standard slope gradient and sites where revegetation will not eradicate the possibility of surface failure. The target slopes are those with a gradient which is gentler than 1:1.0. The use of anchor pins of some 50 – 100 cm in length at nodal points of the frame is desirable to prevent sliding of the frame (see Fig. 3-4-6) while protecting the inside of the frame, which is backfilled with good quality soil, with vegetation.

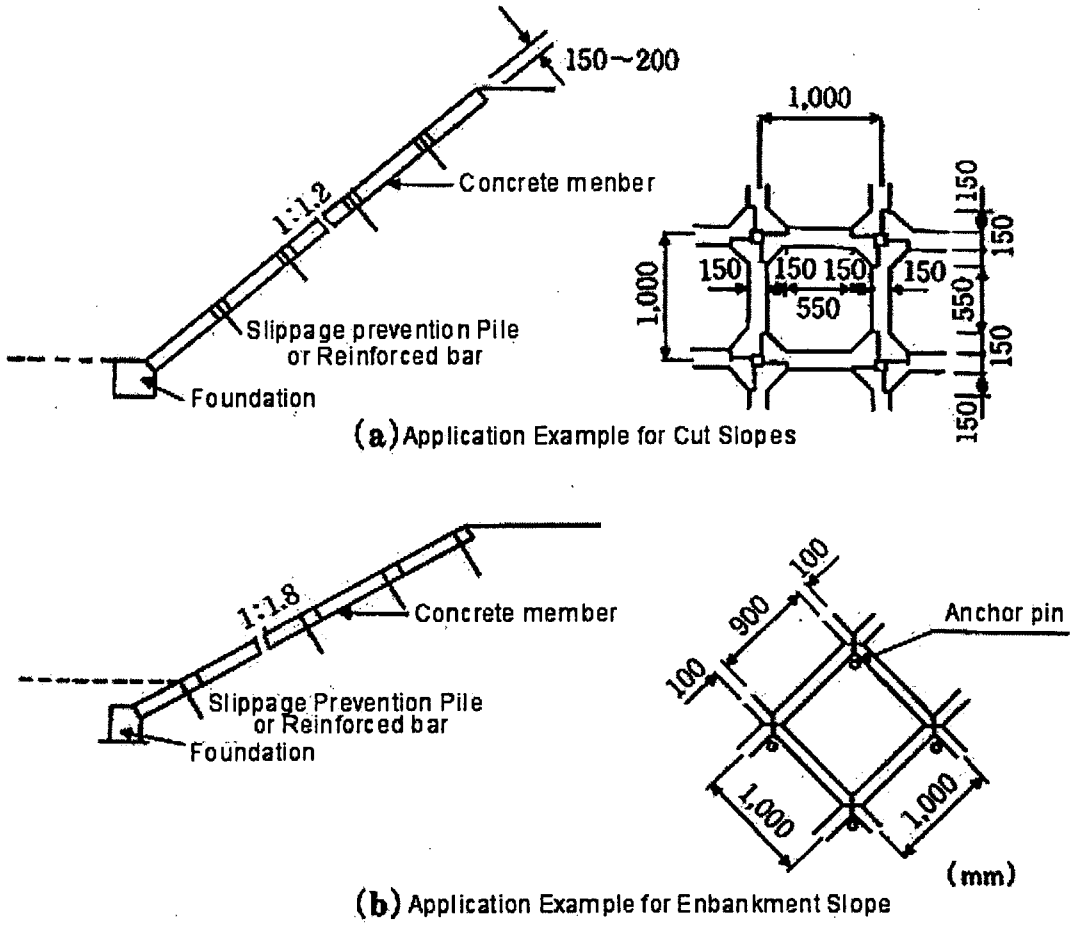


Fig. 3-4-6 Example of Concrete Block Cribworks

(5) Cast-in-Place Concrete Cribworks

This is used when the long-term stability of the slope is questionable or when concrete block cribwork is likely to collapse on a slope made of weathered rock accompanied with spring water or on a large slope.

And also, this type of work plays a supporting function when loose stones cannot be fixed by concrete spraying on bedrock which has many joints and cracks. Moreover, it may be used independently or jointly with ground anchor works to prevent slope failure.

Frames are made of cast-in-place reinforced concrete, and the spaces inside the frames are filled with and protected by stone pitching, concrete block pitching, concrete pitching, mortar spraying or sodding depending upon the conditions.

Cast-in-place concrete cribworks offer superior bending strength than concrete block cribworks because of the beam structure using reinforcing bars. Anchor bars to stop the sliding of the cribworks are introduced at nodal points of the frame depending on the slope conditions (see Fig. 3-4-7).

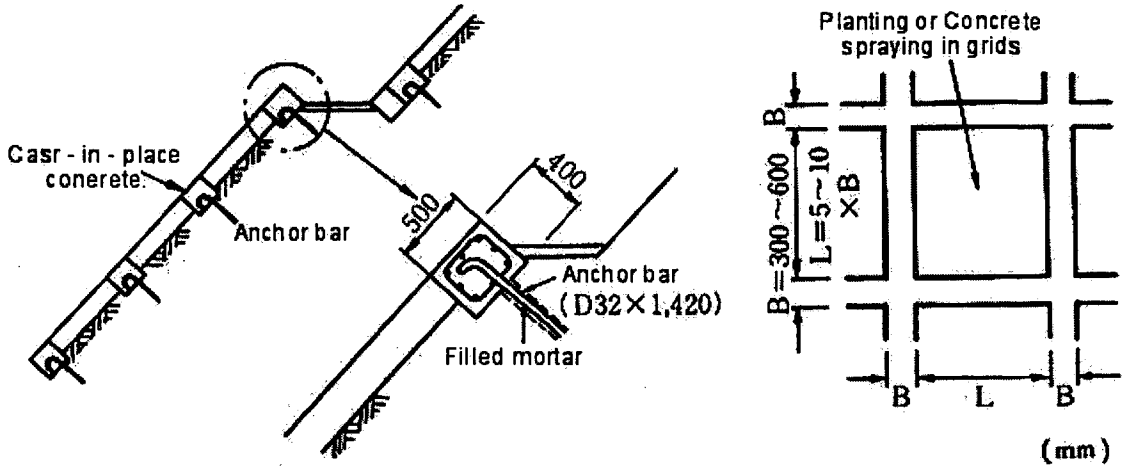
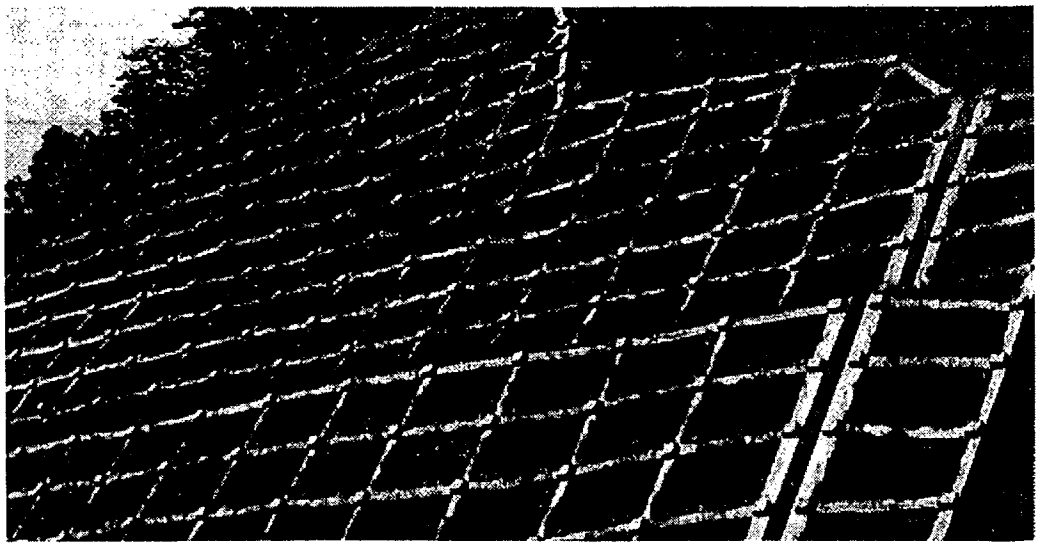


Fig. 3-4-7 Example of Cast-in-Place Concrete Cribworks

(6) Shotcrete Cribworks

Shotcrete cribworks are used for slopes consisting of bedrock with many cracks or slopes of which the protection is urgently required. While the standard function of spray cribworks are the same as those of cast-in-place concrete cribworks, the good workability means that spray cribworks can be applied for slopes with an uneven slope surface. Moreover, the shape of cribworks can be flexibly changed in accordance with the slope conditions (see Photo 3-4-1).

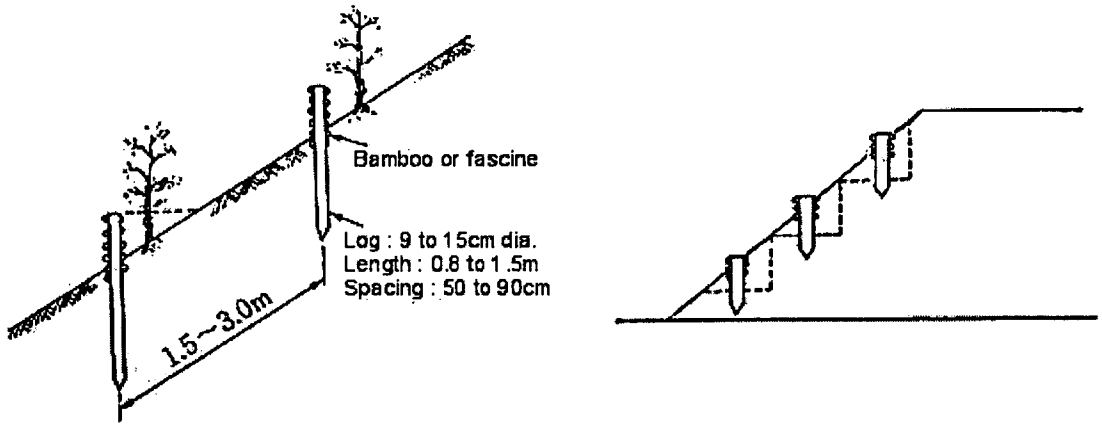


Photograph 3-4-1 Example of Spray Cribworks

The volumes of cement and water for the spraying admixture are determined to achieve a design standard strength of 15 N/mm^2 (150 kgf/cm^2) or higher using a flow value of 120 mm as the yardstick for strength as in the case of (3) – Mortar and Concrete Spraying.

(7) Net Hurdling

Net hurdling is often used to prevent outflow of soil from the slope surface until the plants are fully grown. The net is made by fagot, bamboo, high polymer materials or others to wooden and fixed by stakes driven into the ground to retain the soil (see Fig. 3-4-8).



(a)Top portion of net hurdling above ground (b) Installation of net hurdling by bench cut

Fig. 3-4-8 Examples of Net Hurdling

(8) Gabion Works

Gabions are used where there is spring water on a slope and soil is likely to run off, where a collapsed portion is to be restored.

Ordinary wire cylinders and wire mat gabions are available as gabions, and ordinary cylinders are mainly used for the removal of spring water in the surface layer of slope, for draining surface water.

Wire Mat gabions are used for restoration work after failures in landslide zones and in places where spring water is present. In many cases (refer to Fig. 3-4-9), they are employed in retaining works rather than slope works.

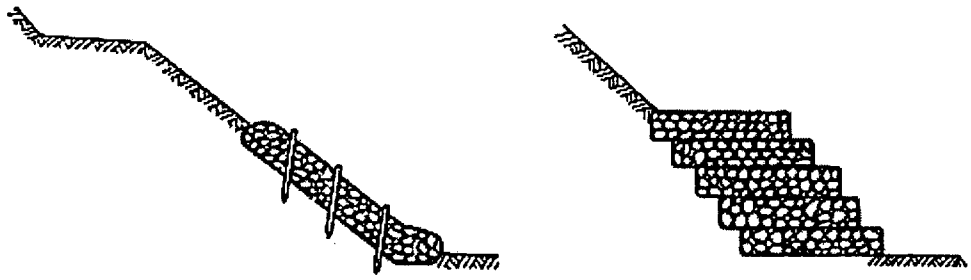
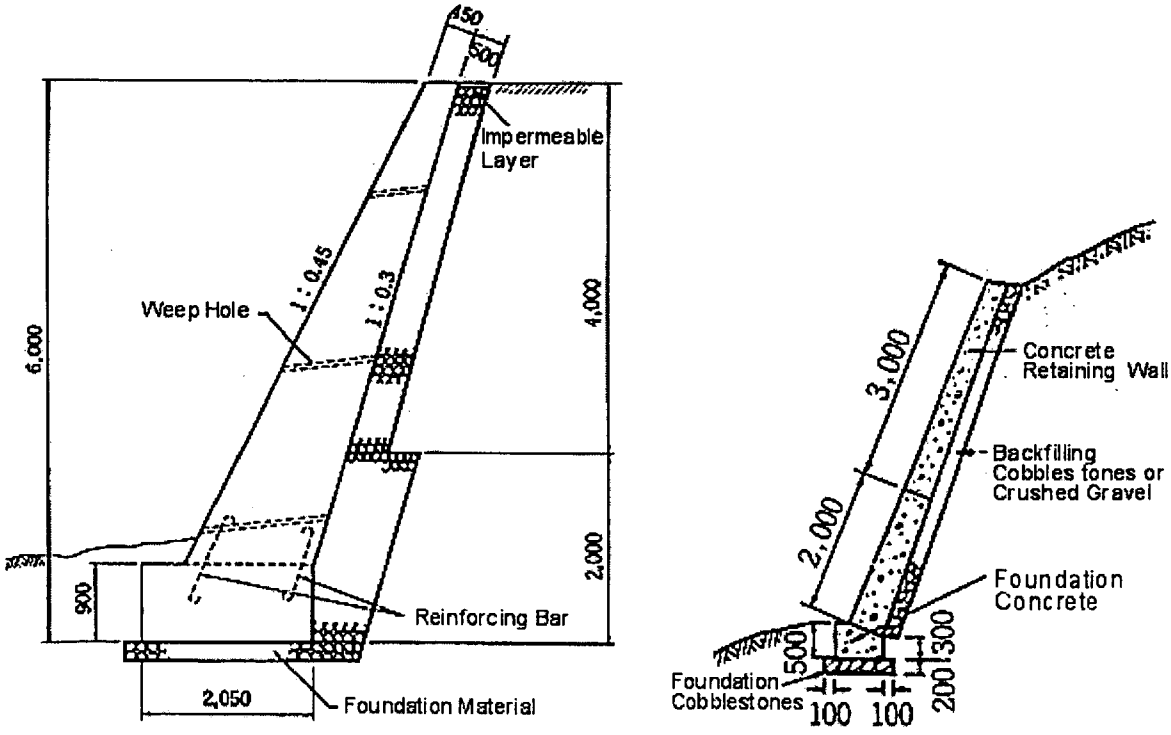


Fig. 3-4-9 Examples of Gabion Works on Slope

(9) Retaining Walls

Retaining walls are constructed for slopes where cutting or banking with a stable gradient is impossible due to the topographical or site conditions or the presence of a water channel or other adjacent structures or facilities (see Fig. 3-4-8 and Fig. 3-4-10).

While the design of a small-scale retaining wall tends to rely on experience without examination of the stability or sliding as well as supporting forces, examination of the stability pursuant to 3.10 – Retaining Walls is necessary for slopes with a height of 5 m or more which have problems in regard to the geological properties and/or foundation ground. There is another type of retaining wall which catches a partial failure of a slope or cut slope at the roadside instead of directly holding down the slope (see Fig. 3-4-11).



(a) Leaning Type Retaining Wall for Embankment (b) Leaning Type Retaining Wall for Cut Slopes

Fig. 3-4-10 Example of Concrete Retaining Wall (Leaning Type)

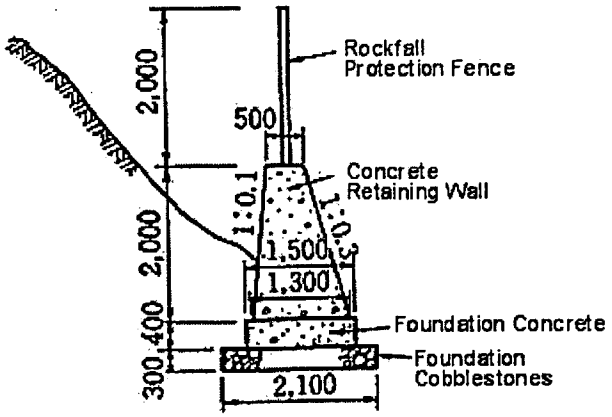


Fig. 3-4-11 Example of Concrete Retaining Wall Used in Combination with Rockfall Protection Fence

(10) Ground Anchor Works

Ground anchor works are usually used to provide suppressive force for slopes which are liable to failure due to joints and/or cracks in the bedrock and slopes which have relatively dense soil but which also have a risk of failure. Ground anchor works can also be used as supporting works for temporary earth retaining walls.

Ground anchor works generally consist of three parts: anchor body, free anchor part and anchor head, as shown in Fig. 3-4-12. As high tensile force acts on the tension member, prestressings steel (prestressing steel bars, prestressing steel strands) is generally used to reduce the relaxation of the steel. For stability analysis for ground anchor works for a slope, calculation of the required anchor force using a stability analysis which assumes circular sliding, straight sliding and compounded sliding is essential (see Fig. 3-4-13).

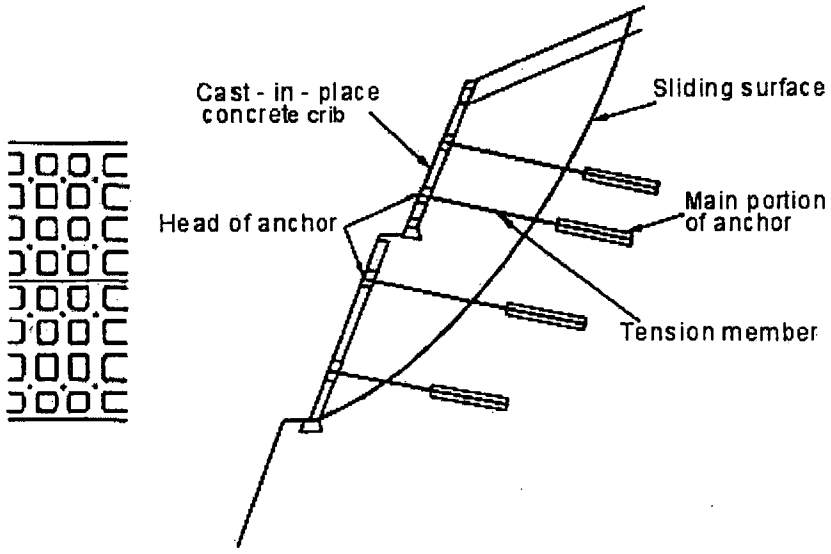


Fig. 3-4-12 Example of Ground Anchor Work

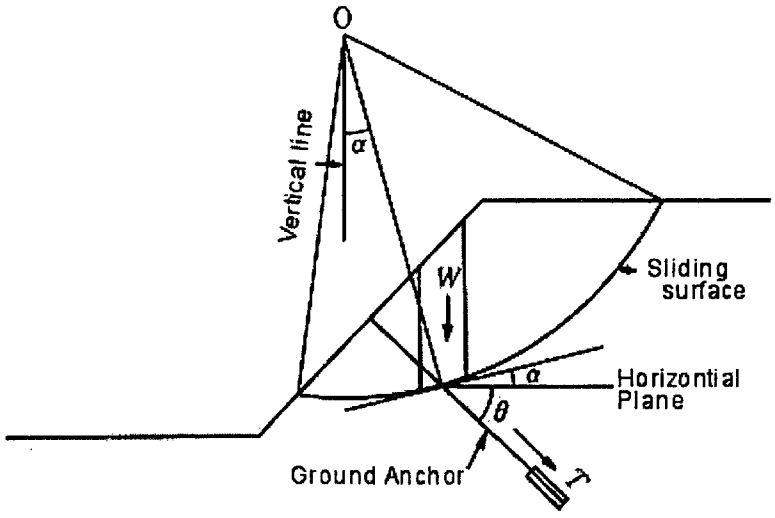


Fig. 3-4-13 Stability Analysis of Slope using Ground Anchor

Ground anchor works are believed to perform the following two functions (see Fig. 3-4-14).

① Increase of the vertical force on the sliding surface to increase the shear resistance force
: Tightening function ($T\sin(\alpha+\theta)\tan\phi$)

② Reduction of the sliding force
: Retaining function ($T\cos(\alpha+\theta)$)

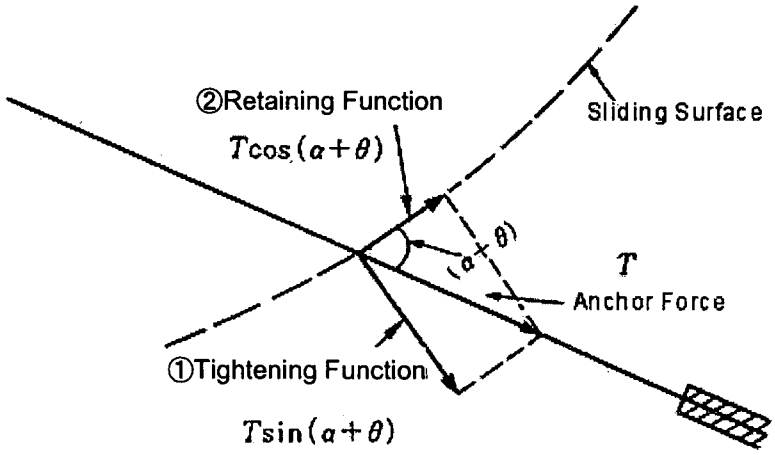


Fig. 3-4-14 Two Functions of Ground Anchor

For the design of ground anchor works to stabilise a slope, an expression which assumes the simultaneous performance of the above two functions is generally used. In certain circumstances, however, the concrete design may reflect the priority given to one of these two functions.

With ground anchor works, bleeding water at the time of grouting often accumulates to reduce the anchorage force. The installing angle for the anchors must, therefore, avoid the range between −10° and +10° from the

horizontal plane. Moreover, the fixed length of the anchors, i.e. the length of the anchor body, should, in principle, be not less than 3 m and not more than 10 m.

When anchors are used as general structures other than temporary structures, they must, in principle, be given double corrosion protection.

(11) Reinforced Earth Works

Reinforced earth works mean a construction method involving the use of various reinforcing materials to stabilise embankment slopes and cut slopes of which the gradient is much steeper than slopes which have no reinforcing material. Reinforced earth works are used for slopes requiring reinforcement because of the impossibility of securing a stable slope gradient for an embankment or cut slope, in turn due to constraints posed by the topography and other factors as shown in the selection flow (Fig. 3-4-1 and Fig. 3-4-2). The reinforcing materials to be used for reinforced earth works for an embankment can have different shapes, including bar, membrane and grid. The actual material to be used can be unwoven geotextile, woven geotextile, netting or synthetic fibres. Reinforced earth works are mainly used for the construction of reinforced embankments and reinforced earth retaining walls, etc. A type of reinforced earth works using continuous long fibres has been developed in recent years (Fig. 3-4-15).

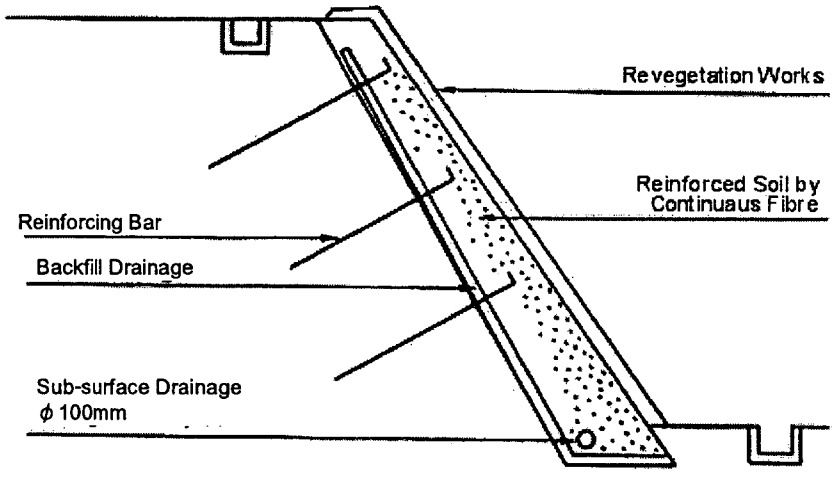


Fig. 3-4-15 Example of Reinforced Earth Works using Continuous Long Fibre

Refer to 3.3.5 – Stability of Embankments for the design of a reinforced embankment and 3.10.8 – Design of Reinforced Soil Walls.

Reinforced earth works for cut slopes (hereinafter referred to as "the cut reinforced earth method") is a method to enhance the stability of an entire slope with the use of reinforcing materials installed into the

ground. This method is used as a measure to prevent relatively small-scale failure, to reinforce a slope with a steep gradient or to reinforce a temporary slope as part of excavation work for a structure (Fig. 3-4-16). The cut reinforced earth method generally involves the use of reinforcing materials (mainly steel bars), grout material, head member and slope protection works as shown in Fig. 3-4-17.

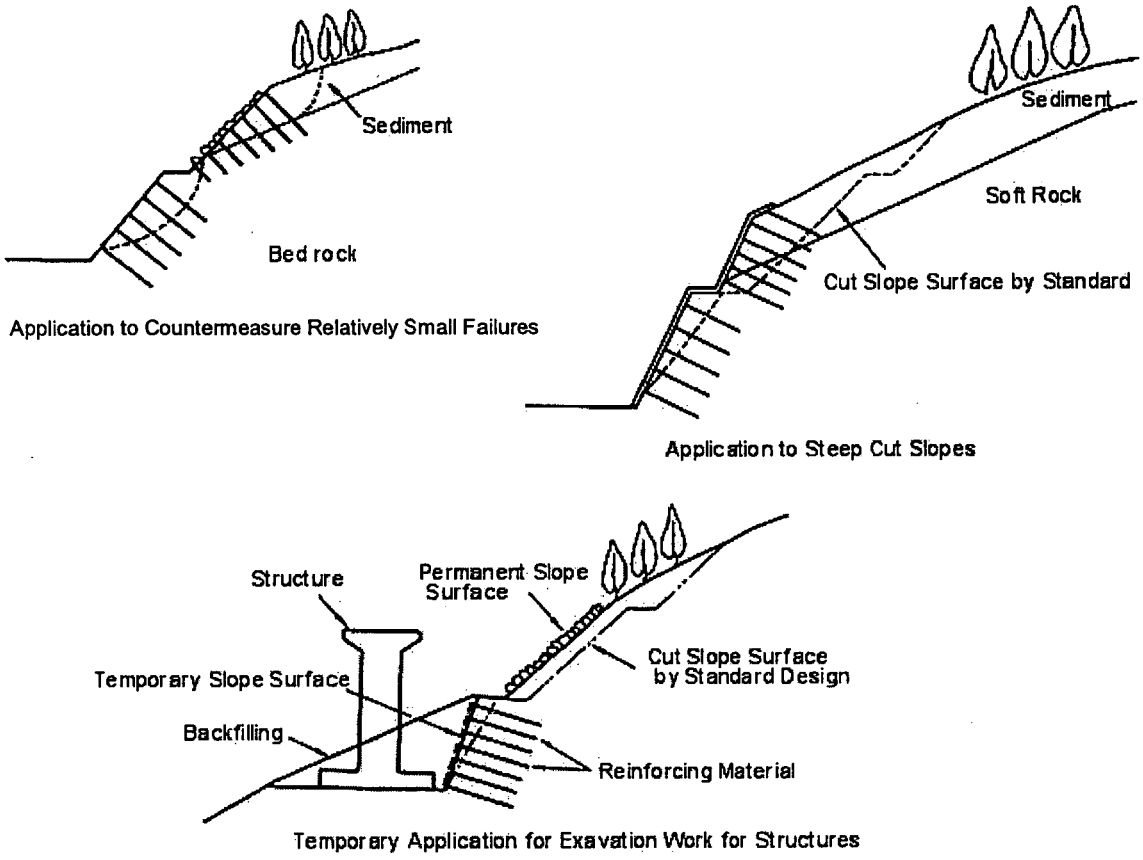


Fig. 3-4-16 Application Example of Reinforced Earth Works for Cut Slopes

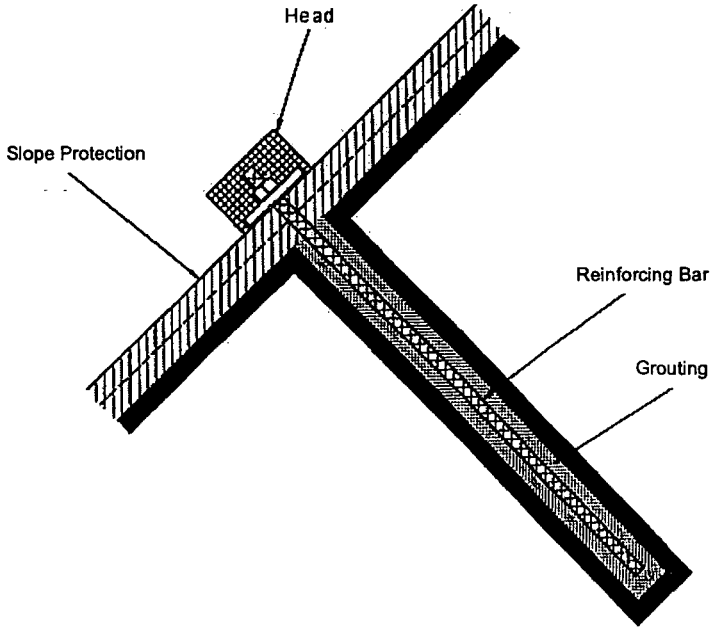


Fig. 3-4-17 Formation of Reinforced Earth Works for Cut Slopes

3.4.5 River Revetment Works

Revetment works are constructed for the main purpose of protecting dykes from erosion at the time of flooding and consist of several components as shown in Fig. 3-4-18.

The conditions to be considered for the design of revetment works include such external forces as the fluid force and earth pressure, etc., changes of the topography near the river course due to changes of the riverbed at the time of flooding, abrasion and damage due to the impacts of flowing sand and gravel, seepage of flowing water and rainwater, natural environment, use of river, workability and construction cost (see Table 3-4-7).

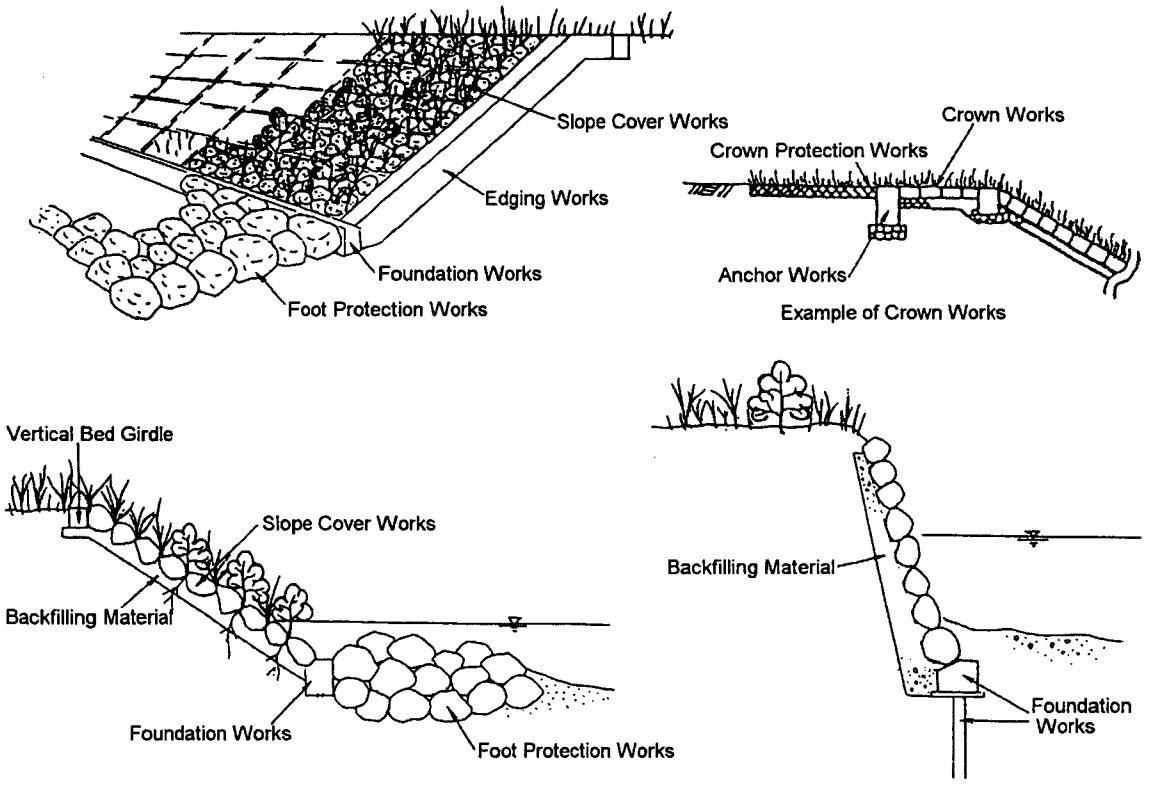


Fig. 3-4-18 Configuration of Revetment Works

Table 3-4-7 Design Conditions for Revetment

Design of Stability	▪ External forces of running water and soil pressure, etc; riverbed fluctuation at the time of flooding ▪ Abrasion, damage and degradation due to the impacts, etc. of flowing sand and gravel ▪ Suction due to the seepage of running water or rainwater
Design of Functions	▪ Prevention/mitigation of erosion ▪ Preservation/improvement of river environment
Design of Rationality	▪ Construction cost and workability

Prior to the design of revetment works, it is essential to study past examples of damage in order to establish a full understanding of the main cause of damage to each component and the characteristics of damage suffered by different types of structures of revetment works as listed below.

① Damage by scouring of the riverbed
② Damage starting from the apron
③ Damage to slope protection works
④ Loss of crown works and crown protection works
⑤ Suction of backfilling soil

(1) Slope Protection Works

As slope protection works are the dominant structural part of revetment works, the design must ensure a stable structure against the impacts of flowing water and driftwood, earth pressure and other forces, taking ecology and landscape into consideration. Although weep holes are not generally introduced for bank protection works, they may be required to combat high residual water pressure in some cases.

Suction prevention materials are used to prevent the suction of the backfilling soil through voids of the slope protection works when residual water behind the bank protection works escapes or when high speed flowing water acts on the bank protection works.

(2) Foundation and Foot Protection Works

The most prominent damage to revetment works occurs when the foundation and slope protection works are damaged by lifting of the foundations as a result of scouring of the riverbed by a flood. When the foundations are damaged, suction of the backfilling soil could occur, resulting in much wider damage. Because of such a prospect, the decision on the elevation of the foundations (embedment depth) is the most important aspect of foundation design.

The elevation of the foundations must be decided by evaluating the deepest riverbed elevation based on past observation results as well as the relevant survey/research results so that lifting of the foundations of the revetment works does not occur despite scouring by a flood. When the required embedment depth is quite deep, the elevation of the foundations can be raised by introducing foot protection works.

The type of foundations should be spread foundations in the case of good ground. Piles or sheet piles are often used for soft ground. Sheet piles are sometimes used at sites with a high ordinary water level and sites where the likely occurrence of scouring must be taken into consideration.

Foot protection works are used to reduce the velocity of the riverwater flow and also to mitigate rapid scouring by means of directly covering the riverbed. As foot protection works are introduced at sites with a high riverwater velocity, sufficient weight to withstand the fluid force, sufficient volume to prevent scouring of the front part of the foundations of revetment works, a high durability level and a flexible structure to follow changes of the riverbed are required. Typical types of foot protection works are shown in Fig. 3-4-19.

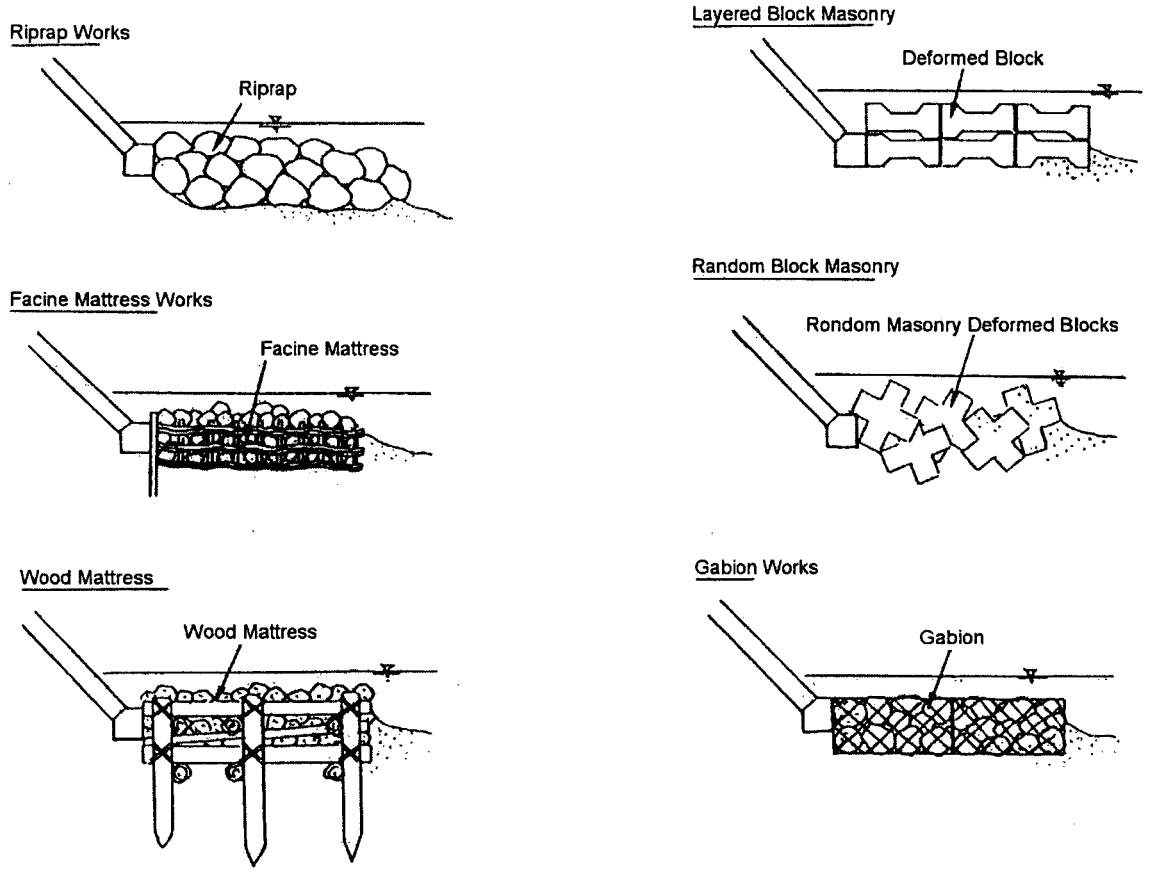


Fig. 3-4-19 Typical Examples of Foot Protection Works

3.5 Drainage

3.5.1 Road Drainage

Drainage is a critical issue for roads. The infiltration of rainwater and/or groundwater to the subgrade or base course can be a major factor for pavement damage while slope erosion due to running rainwater or slope failure due to seepage water destabilises the filled up ground of an embankment. There are many types of damage which are directly or indirectly caused by water, including cut slope failure and damage to retaining walls and other structures due to erosion by rainwater.

Even if water does not cause any structural damage to roads, poor drainage from the road surface can cause traffic congestion or slip accidents by standing water and can also inconvenience pedestrians and people living along roads.

During road construction work, careful consideration of drainage is essential to ensure the trafficability of heavy construction machinery and to properly manage the water content during construction.

There are many road drainage systems as shown in Fig. 3-5-1 and Fig. 3-5-2. Depending on the type of target water, these are basically classified as surface drainage, underground drainage, slope drainage, drainage from the backfill part or the inside of a structure and transverse drainage by a culvert, etc. Fig. 3-5-3 shows the various types of road drainage by purpose.

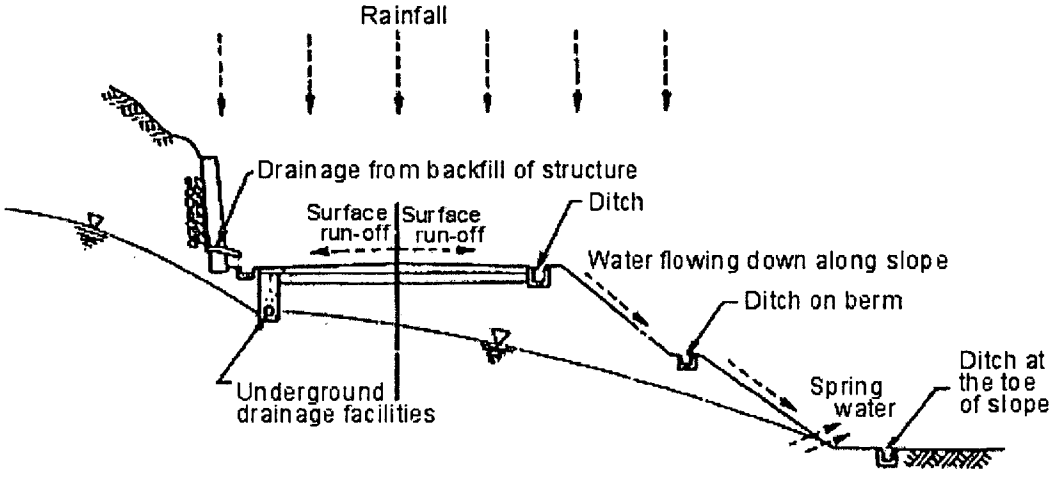


Fig. 3-5-1 Type of Drainage

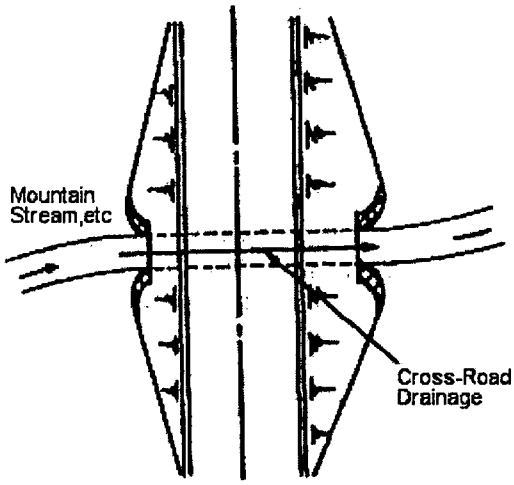


Fig. 3-5-2 Type of Drainage (Transverse Drainage)

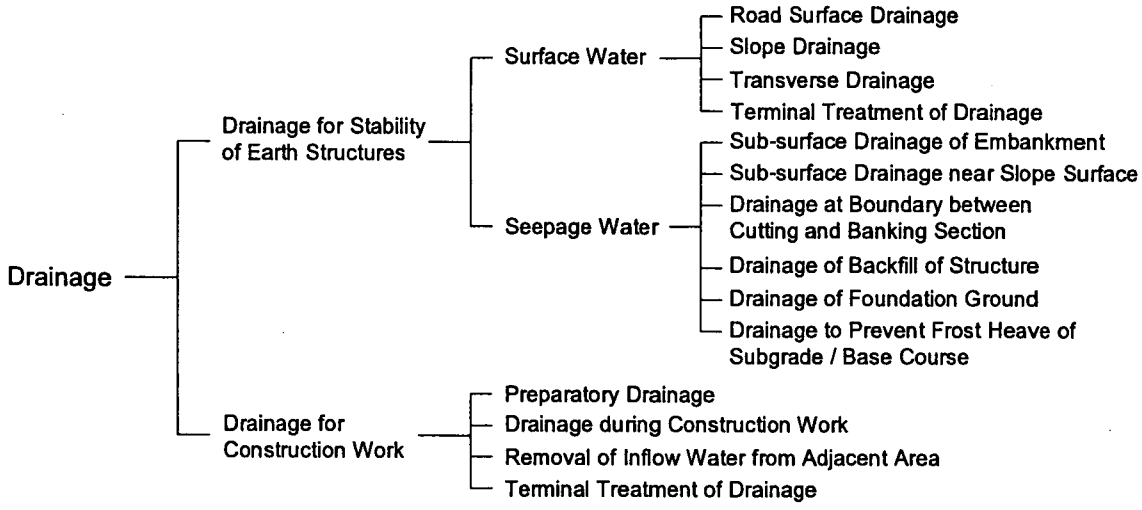


Fig. 3-5-3 Classification of Road Drainage

As the largest factor in the decision on the capacity of drainage facilities is generally the rainwater discharge volume, accurate estimation of the rainfall characteristics is important. In addition, attention must be paid to the frequency of road use, site conditions, anticipated degree of damage by a level of discharge above the design discharge volume and construction cost of individual drainage facilities.

While the discharge volume of rainfall varies depending on the recurrence period of rainfall, Table 3-5-1 and Table 3-5-2 show the model recurrence periods to be adopted as the design standards for drainage facilities.

Table 3-5-1 Selection Standard by Road Classification

Designed traffic volume (vehicles/day) \ Classification of road	National Expressway and Motorway	National Highway	Prefectural road	Municipal road
Larger than 10,000	A	A	A	A
10,000 to 4,000	A	A, B	A, B	A, B
4,000 to 500	A, B	B	B	B, C
Less than 500	—	—	C	C

Table 3-5-2 Standard Recurrence Period of Rainfall Applicable to Drainage Facilities

Class	Level of Drainage Capacity	Recurrence Period of Rainfall	
		(a)	(b)
A	High	3 years	More than 10 years[(c)]
B	Normal		7 years
C	Low		5 years

Note; (a) Applicable to normal road drainage such as road surface or small scale slope
(b) Applicable to important road drainage such as transverse drainages which remove runoff water from large scale natural slope or located at important flat urban area where local drainage is difficult
(c) 30 years is advisable for very important transverse drainage from the viewpoint of road maintenance and operation

3.5.2 Surface Drainage

(1) Discharge Volume

The rainwater discharge volume must be known to decide the drainage facility capacity. Conventionally, the drainage facilities were empirically determined from run-offs due to rainfalls in the past. However, it is desired to calculate the run-off by using the following rational formula:

$$Q = \frac{1}{3.6 \times 10^6} C \times I \times a \qquad \ldots\ldots (3.5.1)$$

where

Q : Run-off (m^3/sec)

C : Coefficient of run-off

I : Rainfall intensity within time of concentration (mm/h)

a : Catchment area (m^2)

A flow chart for the rainwater discharge volume calculation process is shown in Fig. 3-5-4.

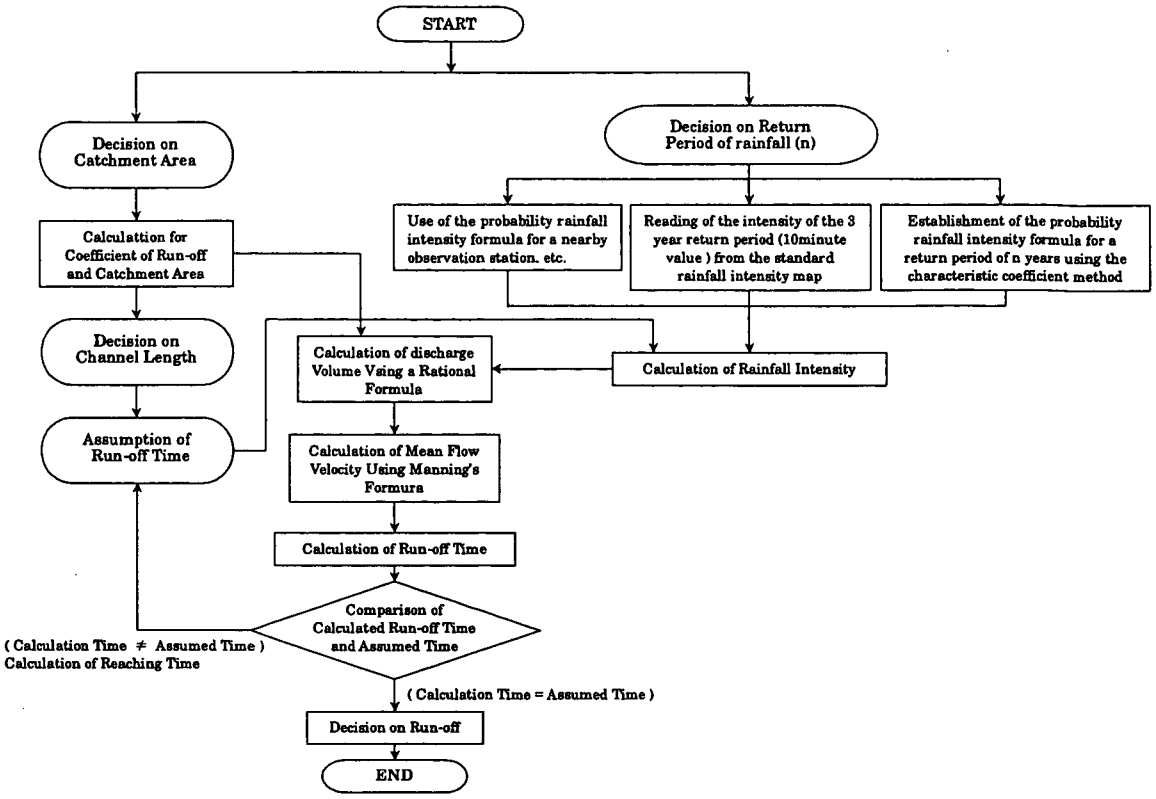


Fig. 3-5-4 Calculation Processes for Run-off

(i) Rainfall Intensity

The rainfall intensity is determined by one of the following methods based on the recurrent period of rainfall selected from Table 3-5-1 and Table 3-5-2.

① Application of the stochastic rainfall intensity formula for a nearby rain observation station

② Use of the standard rainfall intensity map

③ Application of the characteristic coefficient method

For the design of such road surface drainage facilities as side ditches, method ② above should be used. For the design of important drainage facilities, such as determination of the cross-section for water flow for a culvert, etc. which runs across a road, method ① above should be used. However, if reference data on rainfall observed by a nearby rain observation station is unavailable, method ③ should be used.

① Application of the stochastic rainfall intensity formula for a nearby rain observation station

Talbot's formula (3.5.2) shown below is most widely applied for the determination of rainfall intensity.

$$I = a/(t+b) \qquad \text{..... (3.5.2)}$$

Where

I : rainfall intensity (mm/h)

a, b : constant varying by regions

t : Rainfall duration (min.) (time of concentration)

This formula requires rainfall data for arbitrary rainfall duration of t. It is, therefore, necessary to obtain rainfall data from a nearby rain observation station to the planned site for road drainage to assess the probability of occurrence.

② Use of the standard rainfall intensity map

This method involves the use of the standard rainfall intensity map which is prepared from the national map for the 10 minute rainfall intensity with a recurrence period of three years based on the method described in ③ next. As the reference data used for the preparation of the map is mainly based on the data in urban areas, a 20 – 40% increase should be considered in cases where an increase of the rainfall due to topographical factors, such as a mountain area and others, is necessary.

③ Application of the characteristic coefficient method

In the case of this method, only 60 minutes rainfall and 10 minutes rainfall data is used from the wide range of rainfall data to establish the stochastic rainfall intensity formula and its accuracy is known to be virtually the same as the accuracy achieved by the conventional stochastic calculation method. A national rainfall intensity map based on this method has been prepared using 60 minute and 10 minute rainfall data for 25 years at 148 meteorological observatories and observation stations throughout Japan.

(ii) Catchment Area

A catchment area can take different forms depending on the specific purpose of surface drainage facilities. It can consist of (a) only the right of way, (b) both the right of way and neighbouring slope surfaces and/or flat land or (c) a relatively large adjacent area, such as an adjacent swamp.

Facilities such as side ditches belong to either (a) or (b), while transverse drainage facilities, including culverts, belong to (c).

(iii) Time of Concentration

The time of concentration t is classified into the time for water to reach the drainage facilities from the farthest point in the catchment area (inflow time t_1) and the time to reach the design site via a drainage pipe, etc. (downflow time t_2). The design of road facilities should be based on the following expressions.

- Road surface drainage $t = t_1$
- Drainage pipe/culvert $t = t_1 + t_2$

(a) The inflow time is considerably affected by many factors, including the conditions of the ground surface, gradient and size and topographical features of the catchment area. In general, however, it is sufficient to use 15 – 30 minutes for mountain land, 3 – 5 minutes for a cut slope and 5 minutes for an urban area depending on the slope length based on past experience.

(b) The downflow time at a point where it is necessary to calculate the rainwater discharge volume can be approximately estimated by dividing the longest aggregate length of the side ditches and culvert, etc. in the upstream by the mean flow velocity. This mean flow velocity is calculated by Manning's formula (3.5.3).

$$v = 1/n \times R^{2/3} \times i^{1/2} \quad \text{..... (3.5.3)}$$

Where,

v : mean velocity of flow (m/sec)

n : coefficient of roughness (sec/m$^{1/3}$)

R : hydraulic mean depth (m)

(= A/P; A : cross-sectional area for water flow; P : length of wetted perimeter)

i : water surface gradient (or water channel gradient)

In urban areas, the standard mean velocity is 0.5 – 1.0 m/sec for side ditches, 0.6 – 1.0 m/sec for small diameter drainage pipes and 0.8 – 2.0 m/sec for large diameter drainage pipes.

(iv) Coefficient of Discharge

The coefficient of discharge varies depending on the rainfall intensity and characteristics of a specific catchment area and its uniform determination is difficult. As a result, various organizations use their own value. Table 3-5-3 and Table 3-5-4 should be referred to for such drainage facilities as road surface drainage facilities for which the rainfall recurrence period is short while Table 3-5-5 should be referred to for such drainage facilities as culverts for which the rainfall recurrence period is relatively long.

When the land use is complicated, the weighted average value using the component area ratio (P_i).

$$C = \sum(P_i \cdot C_i) \quad \text{..... (3.5.4)}$$

Table 3-5-3 Basic Coefficients of Run-off by Ground surface Condition

Ground surface condition		Coefficient of run-off
Road surface	Pavement	0.70 – 0.95
	Gravel road	0.30 – 0.70
Shoulder, slope, etc.	Fine-grained soil	0.40 – 0.65
	Coarse-grained soil	0.10 –0.30
	Hard rock	0.70 –0.85
	Soft rock	0.50 –0.75
Lawns on sandy soil	Gradient 0 to 2%	0.05 –0.10
	2 to 7%	0.10 –0.15
	more than 7%	0.15 –0.20
Lawns on clayey soil	Gradient 0 to 2%	0.13 –0.17
	2 to 7%	0.18 –0.22
	more than 7%	0.25 –0.35
Roof		0.75 –0.95
Intermediate area		0.20 –0.40
Park with abundant lawn and trees		0.10 –0.25
Mountainous area with gentle slope		0.20 –0.40
Mountainous area with steep slope		0.40 –0.60
Paddy field, water surface		0.70 –0.80
Cultivated field		0.10 –0.30

Table 3-5-4 Average Coefficient of Run-off by Type of Land Use

Type of Land Use		Coefficient of Discharge
Commercial Area	▪ Downtown	0.70 – 0.95
	▪ Adjacent area to downtown	0.50 – 0.70
Industrial Area	▪ Dense industrial area	0.50 – 0.80
	▪ Not so dense industrial area	0.60 – 0.90
Residential Area	▪ Residential area with little open land	0.65 – 0.80
	▪ Apartment area	0.50 – 0.70
	▪ Residential area with much open land/gardens	0.30 – 0.50
Green Area and Others	▪ Park, cemetery	0.10 – 0.25
	▪ Sports ground	0.20 – 0.35
	▪ Railway marshalling yard	0.20 – 0.40
	▪ Paddy field, dry farmland and woods, etc.	0.10 – 0.30

Table 3-5-5 Coefficient of Run-off

Road surface and slope surface	0.70 – 1.0	Urban area	0.60 – 0.90
Steep mountain land	0.75 – 0.90	Forest zone	0.20 – 0.40
Gentle mountain land	0.70 – 0.80	Catchment area of mountain stream	0.75 – 0.85
Undulating land and woods	0.50 – 0.75	Catchment area of small river on flat land	0.45 – 0.75
Flat farmland	0.45 – 0.60	Catchments area of major river of which more than half run flat land	0.50 – 0.75

(2) Surface Drainage Facilities

Proper attention must be paid to the disposal of surface water in order to ensure the stability of earth structures, such as embankments.

Among drainage facilities which target surface water, side ditches are the predominant facilities for road surface drainage while drainage ditches introduced at the top of a slope and berms are the predominant facilities for slope drainage. Side ditches are mainly used to deal with surface water from adjacent land. In the case of a road which crosses the mouth of a valley, the installation of a culvert, which is capable of dealing with the design discharge volume calculated by Formula 3.5.1, across the road may be necessary even if the discharge volume during fair weather is small.

Side ditches or drainage ditches which are used as surface water drainage facilities may be simply dug, lined with turf or made of concrete. Dug ditches are used as provisional facilities. In Japan, permanent drainage ditches which are widely used are made of concrete or asphalt admixture. The cross-sectional form of a concrete drainage ditch can be L-shaped, U-shaped or semi-circular-shaped. Drainage ditches made of precast concrete are also widely used.

The drainage capacity of these side ditches and drainage ditches can be calculated by the following formula using the cross-sectional area for water flow and the mean velocity.

$$Q = A \times v \quad \cdots\cdots (3.5.5)$$

Where,

Q : drainage volume (m^3/sec)

A : cross-sectional area of the water flow section (m^2)

v : mean velocity established by Manning's formula (m/sec)

When the velocity in the drainage ditch is too large, water splash occurs at the bends in the case of an open concrete ditch, possibly leading to the scouring of nearby earth structures. Similarly, scouring of the bottom may occur in the case of dug side ditches or similar simple ditches. When the velocity is too small, sedimentation may occur. These eventualities must be taken into consideration in the design of the longitudinal gradient and cross-section. It is desirable to allow a 20% margin for the cross-sectional area calculated by Formula (3.5.5) in anticipation of sedimentation. A higher margin is necessary for places where a large amount of soil may be discharged at the time of a downpour.

3.5.3 Slope Drainage

(1) Drainage for Cut and Embankment Slopes

Damage of slope due to water can be roughly divided into the surface erosion due to slope surface water and failures due to the increase in pore water pressure or the decrease in shearing strength of earth forming the slope by scouring and water seepage. Fig. 3-5-5 shows the drainage facilities for roadside slopes.

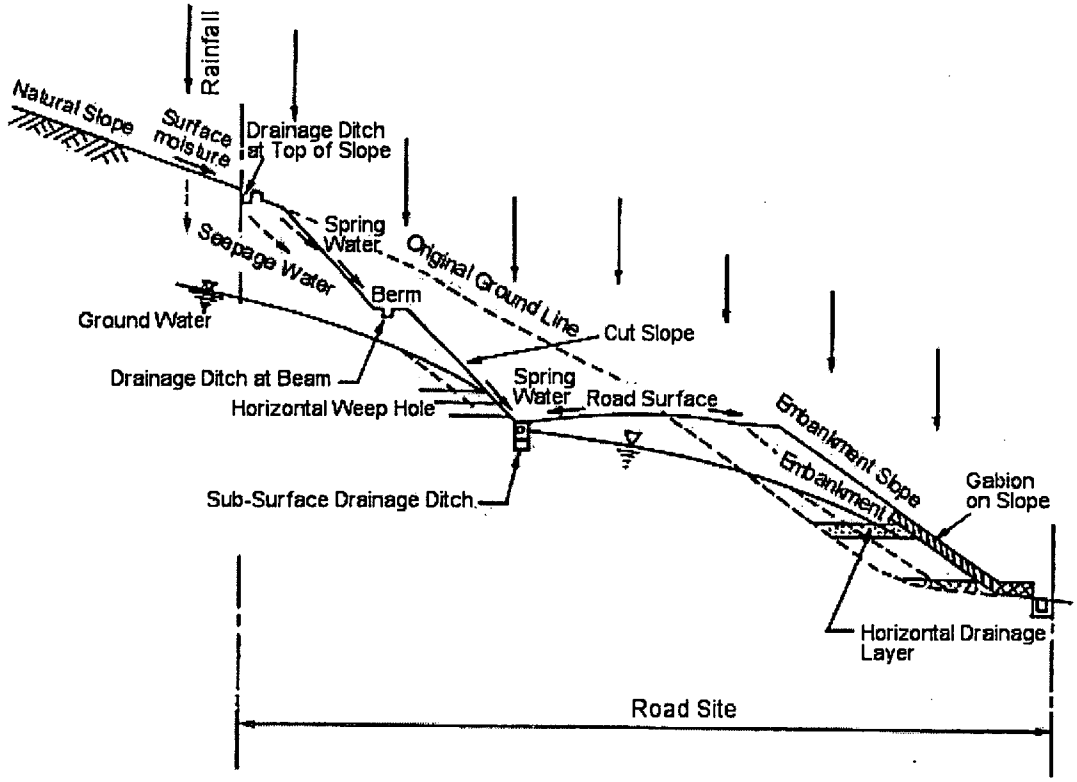


Fig. 3-5-5 Slope of Road and Flow of Water

(i) Cut Slopes

It is required to install drainage ditches, according to circumstances, earth banks or even concrete banks so as to prevent ground water from flowing onto the cut slope from adjacent natural slope in order to avoid any slope failure by changing the direction of water flow or by storing water.

In the case of a large slope, the amount of water running down across the slope may become considerably large at the downstream portion, so that it is required to install drainage facilities on berms to remove water in order to prevent erosion due to the surface water.

For places where spring water is expected to come from cut surfaces or where spring water occurs during rainfalls, the stability should be secured by installing slope protection work such as slope cribwork with horizontal weep holes as drainage.

(ii) Embankment Slopes

According to example of failures after completion of embankment, caution is required shown as following cases.

① Places where rainwater gathers

In the case of curved section of road where the cross grade of the road becomes a super-elevation, and where the surface run-off on the road concentrates on such places as A and B as shown in Fig. 3-5-6 and runs away outside of road when water volume exceeds the drainage capacity of inlets at A and B thereby sometimes resulting in the erosion of slope surface.

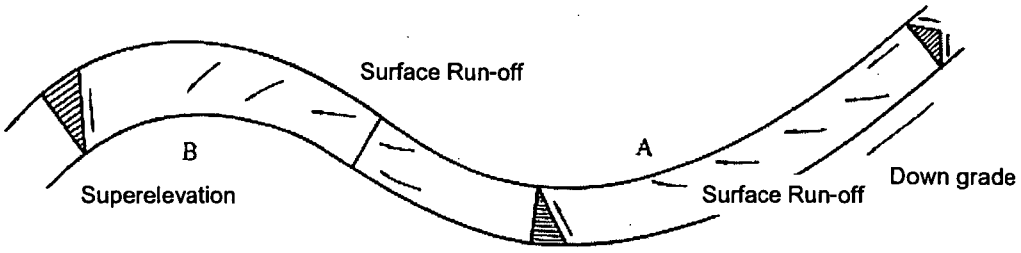


Fig. 3-5-6 Concentration of Water on the Surface of Curved Section

② Half-bank and half-cut section

Rainwater falling on to the cut slope side may not be drained to the side ditch at the bottom of the slope and may cross the road surface to run down the embankment slope, causing scouring there.

③ Places with transverse drainage facilities across the road

A major slope collapse or complete washing away of the embankment may occur due to overflow, in turn caused by an excessive volume of running water beyond the capacity of the drainage facilities across the road (culvert) and/or clogging of the entrance by driftwood and/or sediment.

④ Embankment at the site of poor subsurface drainage in a mountain area

When the gradient of the ground is relatively steep and the seepage water in the ground is poorly drained, deep failure could occur.

(2) Slope Drainage Facilities

The drainage facilities introduced to stabilise slopes target either surface water or seepage water as well as groundwater and the main types are listed in Table 3-5-6.

Surface drainage facilities on a slope are designed to reduce the amount of water running down the slope to prevent slope erosion by surface water and to prevent the infiltration of surface water into the slope. The relevant facilities include a drainage ditch at the top of the slope (Fig. 3-5-7), a vertical drainage ditch and a drainage ditch at the berm (Fig. 3-2-3). In addition, extra consideration, such as the introduction of a downward gradient at the berm, is necessary to prevent the concentration of surface water on the berm to create another downward flow of surface water on the slope surface below the berm.

The destruction of slope drainage facilities is mainly caused by scouring outside or below the ditch by water which has failed to run through the ditch. Those drainage facilities of which the purpose is to collect surface water must have a sufficient depth into the original ground to easily receive running water. Moreover, backfilling should be carefully conducted using impermeable materials. At places where a rapid flow is anticipated, the introduction of certain measures is necessary. These include the use of a lid to prevent water from splashing out and drainage ditch protection using turf or stone pitching to prevent scouring due to splashing water.

In the case where minor spring water is observed at a slope, one good idea is the digging of holes as shown in Fig. 3-5-8 and the insertion of porous piping in these holes. The length of these holes should generally be at least 2 m.

The insertion of a sand layer at an embankment at certain intervals for drainage purposes as shown in Fig. 3-3-13 is sometimes conducted to prevent the failure of an embankment slope.

Table 3-5-6 Types of Slope Drainage

Purpose	Type of Drainage Works	Function
Surface drainage (drainage from road surface, adjacent area and slope surface)	▪ Drainage ditch at the top of a slope ▪ Vertical drainage ditch ▪ Drainage ditch at a berm	▪ Prevention of descending surface water to the slope surface ▪ Guidance of rainwater on the slope surface to a vertical drainage ditch ▪ Guidance of water in a drainage ditch at the top of a slope and in a drainage ditch at a berm to the base of a slope
Subsurface drainage (drainage of seepage water and groundwater from slope surface)	▪ Subsurface drainage ditch ▪ Gabion works ▪ Lateral drainage hole ▪ Vertical drainage hole ▪ Horizontal drainage layer	▪ Drainage of groundwater and seepage water from the slope surface ▪ Reinforcement of the base of a slope along with a subsurface drainage ditch ▪ Drainage of spring water from the slope surface ▪ Drainage of seepage water from the slope surface through a drainage well ▪ Drainage of water from an embankment or seepage water from the ground to an embankment

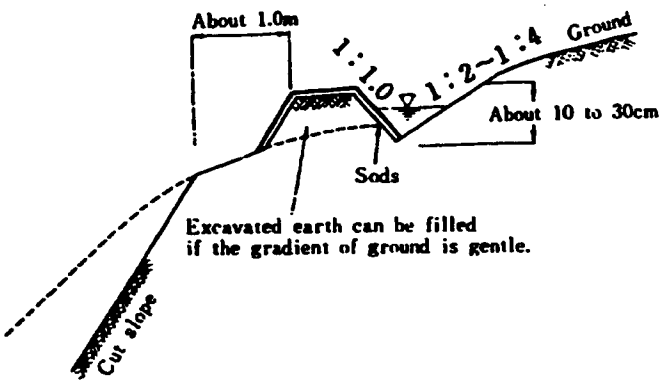


Fig. 3-5-7 Unsupported Gutter

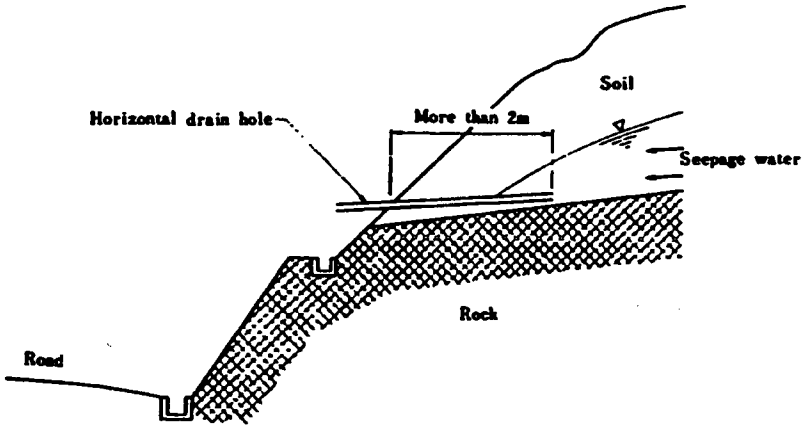


Fig. 3-5-8 Horizontal Weep Holes

3.5.4 Subsurface Drainage

(1) Purposes of Subsurface Drainage

A decline of the shear strength of soil due to an increase of the water content is often experienced. In short, the shear strength of unsaturated soil declines in line with an increase of the water content. The shear resistance of soil generally declines with an increase of the pore water pressure because of the decline of the effective stress.

When this situation occurs at the subgrade or base course, the pavement will be significantly damaged. For example, mud pumping through the joints or cracks of the concrete pavement occurs when the base course is saturated due to poor subsurface drainage. When a similar phenomenon occurs in the case of an asphalt pavement, mud will flow into minute cracks on the surface layer, preventing the closure of these cracks. As a result, damage to the pavement will considerably accelerate.

To prevent such a phenomenon, it is necessary to conduct subsurface drainage to lower the groundwater level below the road surface in order to shut off or drain seepage water into the subgrade, base course and/or filled up ground from adjacent areas of the road as well as the road surface.

(2) Subsurface Drainage Facilities

It appears that subsurface drainage is designed based on previous work which took place at sites with similar conditions instead of conducting new calculation in many cases. However, examination of the seepage flow based on survey data is necessary for important drainage facilities.

The drainage facilities used for subsurface drainage include sub-surface drainage ditches, transverse sub-surface drainage ditches and an impermeable layer. When groundwater inflows from only one side of a road at sloping land, a sub-surface drainage ditch is introduced at the road side near the mountain as shown in Fig. 3-5-9-(b). When the groundwater surface is virtually flat at flat land, a sub-surface drainage ditch is introduced on both sides of the road (Fig. 3-5-9-(a)). An additional ditch is required below the median when the road width is very large (Fig. 3-5-9-(c)).

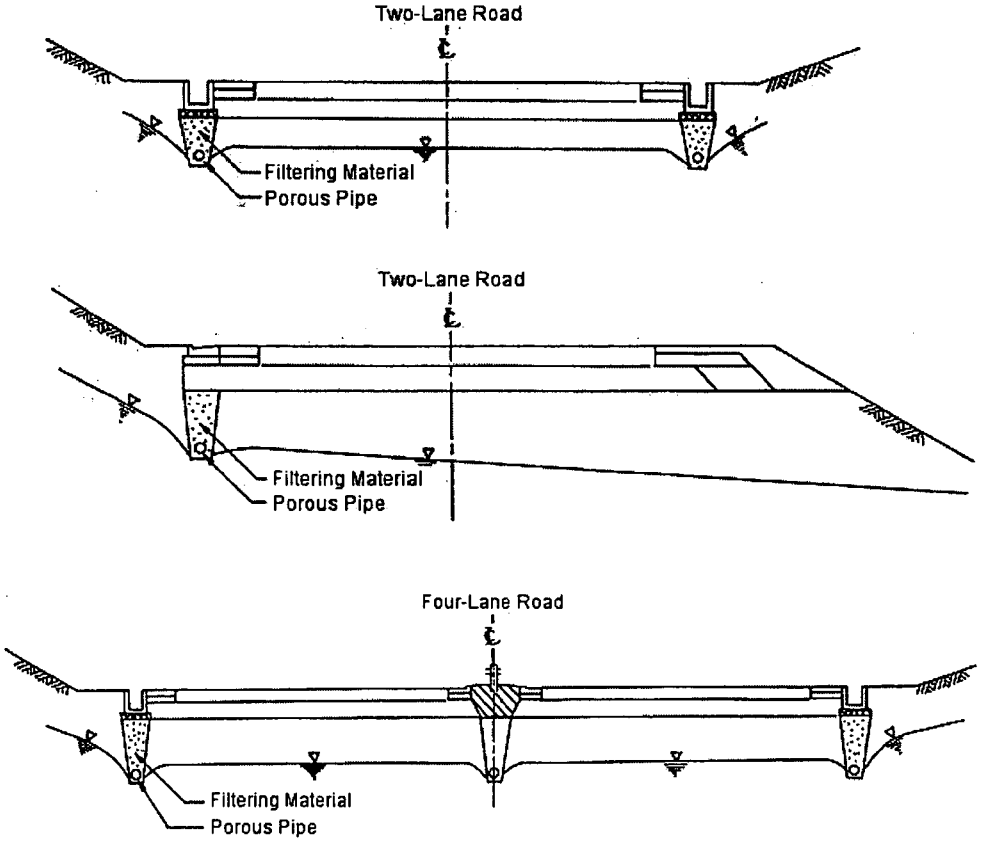


Fig. 3-5-9 Sub-Surface Drain Ditch

In areas with particularly abundant groundwater, sub-surface drainage alone is insufficient. A horizontal impermeable layer is, therefore, introduced at the boundary between the subgrade and base course or inside the subgrade or filled up ground to guide the seepage flow into a sub-surface drainage ditch.

In the case where it is necessary to reduce the inflow of groundwater, which is bolstered by much seepage water from a cut slope, to the embankment at half-bank and half-cut section of a road or the longitudinal boundary between the cutting section and banking section of a road, the sub-surface drainage ditches shown in Fig. 3-5-5, 4-3-5 and 4-3-6 should be introduced.

In many cases, a depth of some 1.0 – 2.0 m is required for roadside sub-surface drainage ditches. However, the actual requirement varies depending on the topographical and geological conditions and the groundwater level. In principle, a drainage pipe should be installed at the bottom of a sub-surface drainage ditch (Fig. 3-5-10). Although a porous concrete pipe is often used as the drainage pipe, other types, such as a permeable pipe, may be used to suit the specific site conditions.

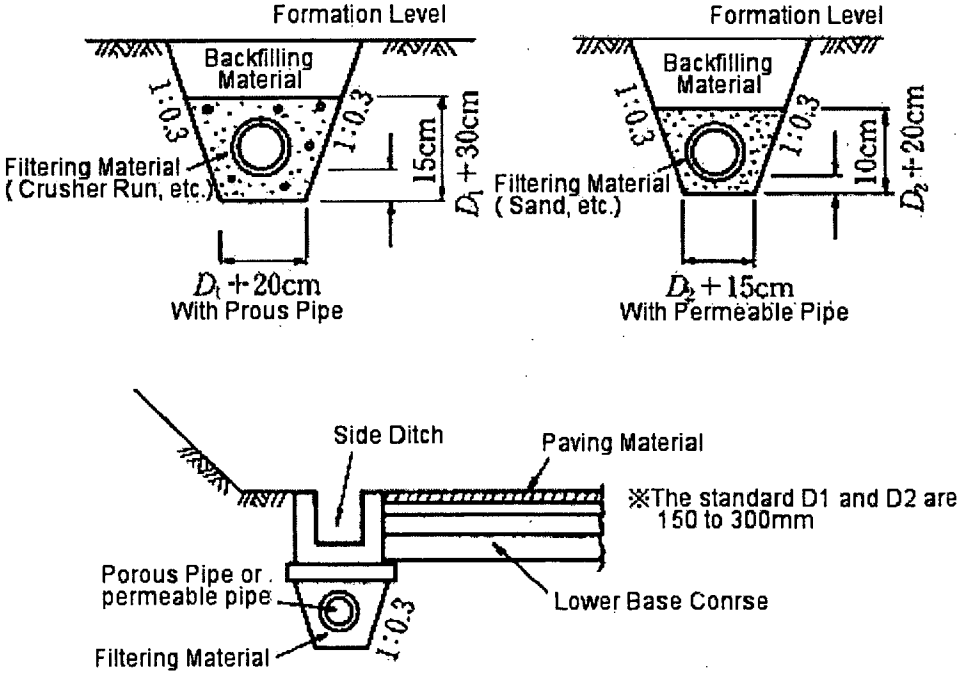


Fig. 3-5-10 Example of Sub-Surface Drainage Ditch

The protection of a porous pipe with a good quality filtering material is desirable to prevent the inflow of soil into the pipe. It is essential for the backfilling material for the drainage ditch to be a highly permeable filtering material which is capable of preventing the inflow of minute soil grains from both sides of the ditch.

The requirements for filtering materials are high stability of the grains to resist weathering or dissolution into water and an appropriate particle size distribution curve. The particle size distribution curve required must indicate that the selected filtering material prevents the clogging of a porous pipe by the inflow of the subgrade or base course soil and provide sufficient permeability compared to the subgrade soil. It is, therefore, desirable for filtering materials to satisfy the following conditions.

$$\frac{D_{15}(filtering\ \ material)}{D_{85}(subgrade\ \ soil)}<5$$

$$\frac{D_{15}(filtering\ \ material)}{D_{15}(subgrade\ \ soil)}>5 \qquad \text{..... (3.5.6)}$$

$$\frac{D_{85}(filtering\ \ material)}{D(pore\ \ diameter)}>2$$

Here, D_{15} and D_{85} are the particle sizes corresponding to 15% and 85% of the pass percentage by weight respectively in the grain size distribution curve.

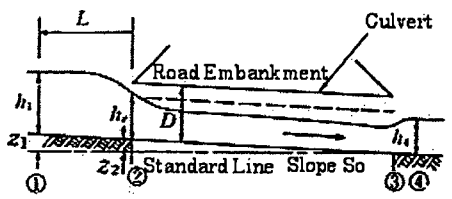
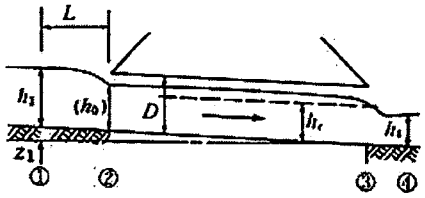
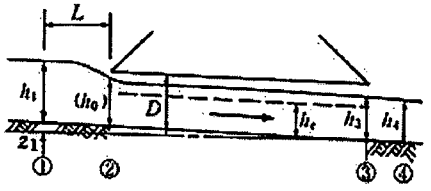
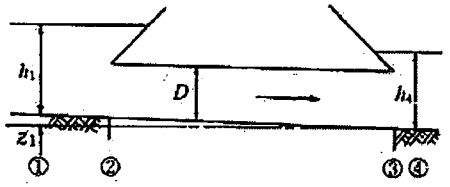
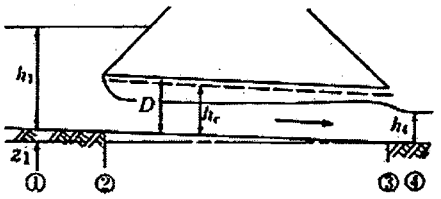
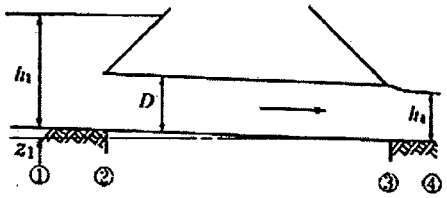
3.5.5 Transverse Drainage across Road

When a road crosses an existing water channel or mountain stream, etc., drainage facilities are introduced across the road. While such facilities usually take the form of a culvert, a bridge may be constructed when the span is very large or when it appears that a culvert will be insufficient to perform the required drainage function.

The cross-section of a culvert for water flow must be large enough to safely allow the passing of the design discharge volume. The value obtained by Formula (3.5.1) should be used as the design discharge volume for a culvert.

In principle, the gradient, bottom elevation and width of a culvert should be identical as much as possible to those of the existing water channel to prevent sedimentation or erosion. However, when the introduction of a culvert is planned at a mountain stream with a very steep riverbed gradient, a culvert gradient within approximately 10% is desirable to ensure workability and to prevent its slippage and abrasion by soil.

When the width of a culvert is narrower than the width of the water channel in the upstream, sudden narrowing of the water channel width raises the water level immediately upstream of the culvert. It is, therefore, desirable for the width of the entire culvert or at least its entrance to be as wide as possible so that the culvert can smoothly be attached to the water channel in the upstream.

Type	Hydraulic Condition	From of Flow	Remarks
1	The critical depth occurs at the entrance. $D > (h_1 + z_1 - z_2)/1.5$ $h_4 < h_c$ $S_0 > S_c$		Measures to prevent scouring are necessary against hydraulic jump may occur near the exit.
2	The critical depth occurs at the exit. $D > (h_1 + z_1 - z_2)/1.5$ $h_4 < h_c$ $S_0 < S_c$		The introduction of scouring prevention measures is desirable at the exit.
3	Gentle flow throughout (subcritical flow) $D > (h_1 + z_1 - z_2)/1.5$ $h_c < h_4 \leqq D$		The most desirable condition for desiging.
4	Full capacity flow throughout (the exit is submerged) $D < h_4$		This may occur as an extraordinary event but should not be used for designing.
5	Rapid flow at the exit $h_4 \leqq D \leqq (h_1 + z_1 - z_2)/1.5$		ditto
6	Full capacity flow throughout (free discharge at the exit) $h_4 \leqq D \leqq (h_1 + z_1 - z_2)/1.5$		ditto

Note: Symbols
D: culvert height; h: water depth; z: riverbed elevation; S: riverbed gradient; hc: critical depth; Sc: critical

Fig. 3-5-11 Classification of Water Flow at Culvert

The situation of water flow at the site of a culvert is affected by not only the discharge volume and conditions of the culvert but also by the conditions of the water channel in both the upstream and downstream. This situation is generally classified into six types as shown in Fig. 3-5-11.

Types 4 through 6 must be avoided at the design stage as they are both hydraulically inferior and liable to cause serious adverse impacts on an embankment by means of scouring, seepage water and overflow due to the accumulation of water at the upstream side of the road embankment.

The cross-sectional area of a culvert should be designed using a suitable formula for each type after satisfying the hydraulic conditions to make the water flow belong to Type 1 through Type 3 in Fig. 3-5-11. At the same time, the height (or diameter) or culvert D should be determined to meet the following conditions.

① The water surface does not touch the upper surface of the culvert.

② The water depth in the upstream of the culvert does not exceed 1.5 times of the height of the culvert.

$$D > (h_1 + z_1 - z_2)/1.5$$

③ The water level in the upstream of the culvert does not exceed the height of the embankment.

h_1 < (embankment height)

Above conditions ① and ② are necessary conditions for Types 1 through 3 to be viable while condition ③ indicates the requirement to ensure that the raised water level in the upstream does not flow over the embankment.

3.5.6 Drainage during Construction Work

Preparatory drainage during construction work and temporary drainage at an embankment, cut slope and borrow pit are the key to the successful construction work. As lax temporary drainage may necessitate an extension of the construction period and/or the replacement of materials, careful attention must be paid to preparatory and temporary drainage not only at the time of planning but also at the design stage of earthwork.

Particularly in the case of a large-scale cut slope or embankment or a large-scale borrow pit, excavation or banking work could change the direction of water flow and/or the catchment area, resulting in adverse impacts on the existing water channel or river. In view of such a possibility, the advance examination of such disaster prevention measures as drainage ditches and a sedimentation basin depending on when ever such measures are deemed necessary is essential.

3.6 Countermeasures for Soft Ground

3.6.1 Settlement and Stability of Foundation Ground for Embankment

(1) Settlement

In the case of an embankment with a limited width, such as a road, two types of settlement occur, i.e. settlement where only the shape of the ground changes mainly due to the constant volume shear of the ground and the other is settlement due to the volume reduction of the ground through consolidation. Fig. 3-6-1 shows the relationship between settlement and time following the construction of an embankment. The $S-t$ curve in Fig. 3-6-1-(a) indicates the relationship between settlement S and $\log t$ for time at the centre of an embankment when an embankment with the base width B shown in (b) is raised up to the height of H_E during a time period of t_0. In other words, the levels of settlement up to the times of t_0, t_1 and t_2 from the start of banking work are represented by S_0, S_1 and S_2 respectively. S_f indicates the level of final settlement. The type of settlement which only involves a change of the shape increases in line with the progress of banking work but hardly increases after the completion of the embankment. Because of this, it is called immediate settlement. When immediate settlement at the time of the banking height of H_E is given S_i, settlement S_2 at the time of t_2 is given by the following formula as shown in Fig. 3-6-1-(a) and (b).

$$S_2 = S_i + S_c \quad \cdots\cdots (3.6.1)$$

Here, S_c is the level of consolidation settlement which has occurred up to the time of t_2.

As the level of settlement at the time of t_1 and t_2 while the banking height remains the same H_E are S_1 and S_2 respectively as shown in Fig. 3-6-1-(a), the residual settlement of ΔS in the post pavement period up to t_2 is shown by the following formula provided that the pavement work is completed at t_1.

$$\Delta S = S_2 - S_1 \quad \cdots\cdots (3.6.2)$$

It is generally acknowledged that the construction of a high embankment above ground with a thick soft layer causes continual settlement over a long period of time. In such a case, the analysis of residual settlement is highly desirable.

Immediate settlement following a change of the shape of the ground is often ignored because of the negligible quantity in the case where the embankment base B is large and the soft layer is shallow or where the ground strength is high.

In addition to settlement by the embankment load described above, the settlement of an embankment must also take settlement by the pavement load, settlement by the traffic load, settlement by lowering of the groundwater level and/or compressive settlement of the embankment itself into consideration depending on the need.

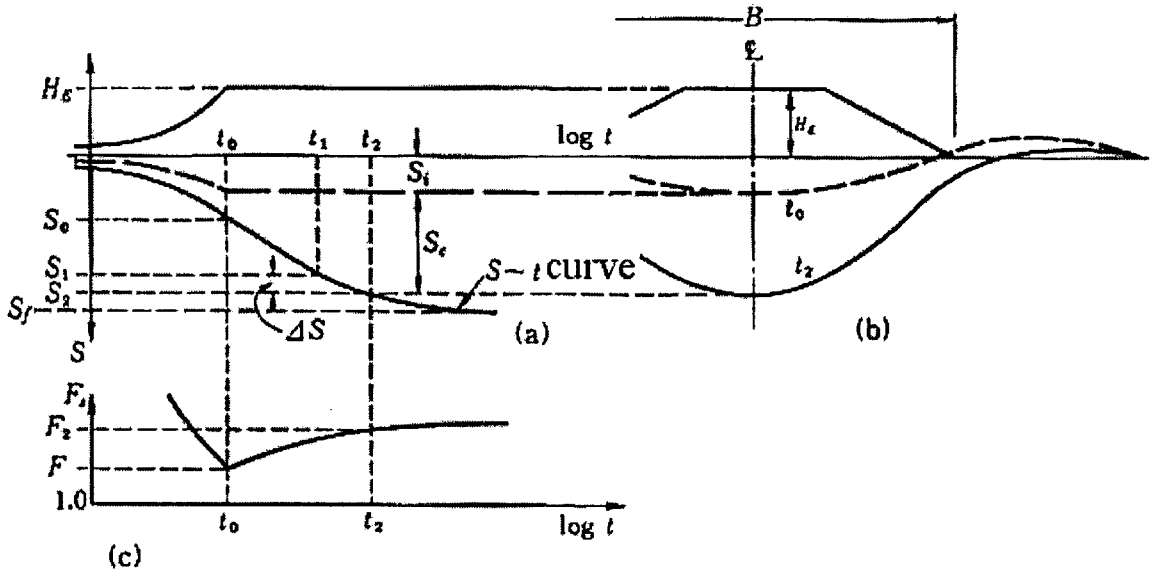


Fig. 3-6-1 Settlement and Stability of Foundation Ground for Embankments

(2) Stability

With the increasing height of an embankment, the ground below an embankment settles and side deformation raises the ground at the sides of the embankment. The scales as well as scope of settlement and upheaval are substantially increased with a rise of the embankment. When the ultimate height of an embankment is reached, sliding failure involving the embankment occurs with the foundation ground. The stability of an embankment may be considerably reduced, especially in such cases as thick soft ground, ultra-soft ground, clayey ground with a high sensitivity ratio and tilted basement layer. In these cases, careful examination of the embankment stability is required.

Fig. 3-6-1-(c) shows the change of the safety factor F_s to sliding failure in time for banking work at a constant speed. According to this figure, the safety factor declines to F when the embankment height reaches H_E. The safety factor gradually recovers, however, in line with the increased strength due to consolidation of the ground when the embankment height stays at H_E. This means that when the construction of a high embankment is planned, slow banking work in line with the consolidation speed of the ground is preferable to rapid banking work. At those sites where important structures exist nearby, where the design safety factor is not particularly high or where abnormal ground behaviour is observed, careful

observation of the ground behaviour (field observation) is essential so that the latest information produced by this observation can be used to predict future ground behaviour and/or to adjust the work speed.

3.6.2 Determination of Design Conditions

In many cases, the soil layer constituting soft ground is not uniform and its properties show significant changes in both the horizontal and vertical directions. It is practically impossible to incorporate very complicated soil information into calculations.

Accordingly, as part of the determination of the ground conditions, the soil parameter to be used for design purposes must be decided by inferring the state of the entire soft ground based on a series of soil investigation results, judgement of the suitability of soil test results and careful selection of an appropriate value. The procedure to determine the ground conditions for design purposes is roughly explained next.

(1) Soil Layer Classification

Firstly, abnormal values among the soil test results are identified in consideration of the mutual relationship of various values and unsuitable test results are either removed or corrected. A calculation model which simplifies the state of ground configuration as shown in Fig. 3-6-2 is then prepared for each cross-section to be examined. The items to be clarified in the soil layer classification process are the thickness of the soft layer (location of the basement layer, i.e. firm layer below the soft layer), boundary conditions in the lower section of the soft layer (where the basement layer is on an impervious layer or pervious layer) and stratification of the soft layer (cross-sectional stratification of the soft layer based on different soil properties). Any judgement relating to soil layer classification must be based on comprehensive analysis of the boring, sounding, soil test and field survey results as well as the topographical and geological analysis results.

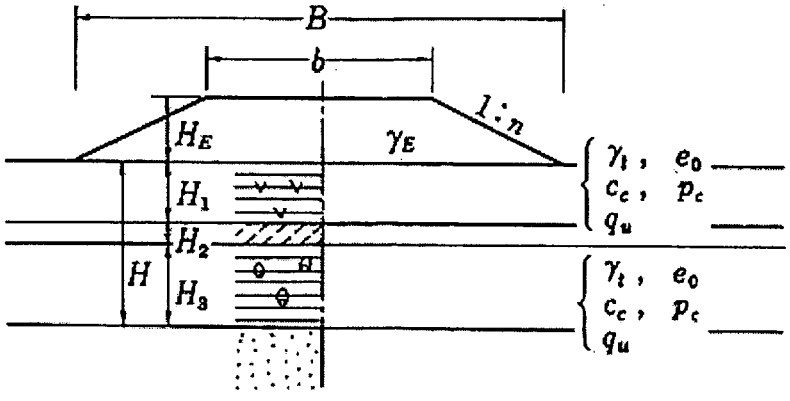


Fig. 3-6-2 Classification of Soil Layer

(2) Determination of Soil Parameter

The numerical values used for stability analysis and settlement calculation for each classified soil layer are, in principle, such typical values as the mean values of the soil test results. In some cases, however, it may be desirable to place emphasis on typical values which incorporate a judgement by an engineer. Some of the concrete methods are explained below.

1) Initial void ratio e_0; consolidation yield stress p_c; overburden pressure p_0; compression index C_c; wet unit volume weight γ_t; unconfined compressive strength q_u

As shown in Fig. 3-6-3-(a) – (e), these parameters are used in the form of typical values, i.e. mean values for each layer in the direction of the depth. However, the overburden pressure p_0 is calculated at the central depth of each layer, taking the groundwater level into consideration.

Half of the unconfined compressive strength q_u is commonly used as the undrained shear strength of c_u of the soft layer for stability analysis. However, in the case where the value of q_u increases with the ground depth, a value which gradually increases with the depth should be used instead of adopting a constant value for each layer.

2) $e - \log p$ curve and consolidation coefficient c_v

The curve formed by soil samples showing values near the mean void ratio of a specific layer can be used as the typical $e - \log p$ curve of the layer in question. For more accurate calculation, however, a group of $e - \log p$ curves should be produced and the typical curve for each layer should be determined using the typical e_0, p_c and C_c values mentioned in Fig. 3-6-3 while referring to the shape of this group of curves. The consolidation coefficient c_v should be determined by regarding the $\log c_v - \log p$ curve of soil samples of which the void ratio is near the mean void ratio of each layer as the typical curve.

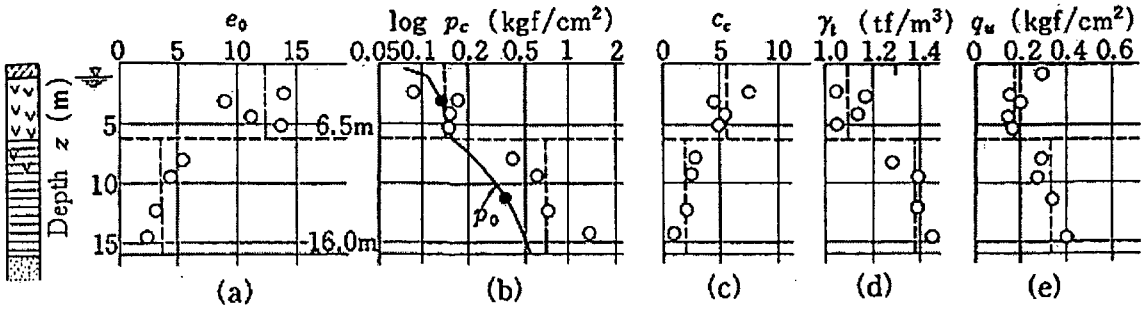


Fig. 3-6-3 Representative Soil Parameters of Each Layer

3) Rate of Strength Increase m

The rate of strength increase m of normally consolidated soil is expressed by the ratio (c_u / p_0) between the undrained shear strength c_u under natural conditions and the overburden pressure p_0 acting on this soil. In general, it is sufficient to select a rate of strength increase from the range shown in Table 3-6-1 depending on the type of soil.

Table 3-6-1 Rate of Strength Increase

Soil Type	Rate of Strength Increase 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

(3) Determination of Work Conditions

1) Shape and Dimensions of Embankment

In general, analysis is conducted solely targetting an embankment up to the design height. In the case of a low embankment, analysis should include the pavement load and traffic load depending on need.

2) Unit Volume Weight of Embankment

Even though it is difficult to clarify the embankment properties at the design stage, the unit volume weight of an embankment can be assumed as shown in Table 3-6-2.

Table 3-6-2 Unit Volume Weight of Embankment

Banking Material	Unit Volume Weight γ_E (tf/m^2)
Gravel; Gravelly Soil	2.0
Sand; Sandy Soil	1.9
Silty Cohesive Soil ($w_L \leqq 50\%$)	1.8
Volcanic Cohesive Soil	1.5

3) Banking Speed

Although it depends on the planned construction period, it is desirable to set the banking speed at below the value shown in Table 3-6-3 when possible.

Table 3-6-3 Banking Speed

Ground Conditions	Banking Speed (cm/day)
Thick clayey ground and muck or peaty ground with thick deposit of organic soil	3
Ordinary clayey ground	5
Thin clayey ground and muck or thin peaty ground with little organic soil	10

3.6.3 Settlement Calculation of Embankment on Soft Ground

(1) Settlement

In general, settlement is estimated by one-dimensional consolidation calculation which considers the settlement of an embankment to mainly be caused by the consolidation of soft ground due to the embankment load while ignoring immediate settlement or by the field observation results of settlement. When settlement of the side ground or loose sand layer due to the construction of an embankment is anticipated to be large, it is desirable to conduct a separate detailed analysis to estimate the settlement.

In the case where settlement due to the changed shape of the ground and settlement of the sand layer, both caused by loading of the embankment, are ignored to regard the total settlement as consisting of only the one-dimensional consolidation settlement of the soft layer below the central part of an embankment, the one-dimensional consolidation settlement S_c for each classified consolidation layer is calculated using Formula (3.6.3). The settlement values calculated in this manner are then totalled to produce the total settlement for the entire soft layer.

$$Sc = \frac{e_0 - e_1}{1 + e_0} H \quad \text{..... (3.6.3)}$$

Where,

e_0 : initial void ratio of the consolidation layer

e_1 : void ratio of the consolidation layer after consolidation

H : thickness of the consolidation layer

In the case where the compressive index C_c or the coefficient of volume compressibility m_v is available for each consolidated layer of a soft layer consisting of normally consolidated soil, the one-dimensional consolidation settlement for each consolidation layer can be calculated by one of the following formulas.

$$Sc = \frac{Cc}{1 + e_0} \log \frac{p_0 + \Delta p}{p_0} \quad \text{..... (3.6.4)}$$

$$S_c = m_v \cdot \Delta p \cdot H \quad \text{..... (3.6.5)}$$

Where,

p_o : overburden pressure before banking

Δp : increased stress by embankment load

(2) Settling Speed

The speed of consolidation settlement is generally calculated based on one-dimensional consolidation which assumes that drainage takes place only along the vertical direction.

Firstly, the effective pervious layer for consolidation drainage is identified based on the state of stratification of the ground and is classified as, for example, the consolidation layers shown in Fig. 3-6-4. Among the soil layers constituting individual classified consolidation layers, the value of c_v of an arbitrarily selected soil layer (for example, c_{v3} for the H_3 layer) is upheld as the typical c_v value of this consolidation layer. The thickness of the layer is then calculated by Formula (3.6.6) by converting all of the layers to a single soil layer with the typical c_{v3} value.

$$H_0 = H_1\sqrt{\frac{c_{v3}}{c_{v1}}} + H_2\sqrt{\frac{c_{v3}}{c_{v2}}} + H_3 \quad \cdots\cdots (3.6.6)$$

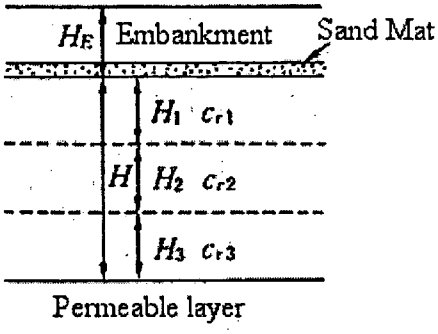


Fig. 3-6-4 Classification of Consolidated Drainage Layer

When the consolidation process progresses through drainage on both sides, half of the converted layer thickness H_0 calculated by Formula (3.6.6) is the longest drainage distance. Accordingly, the typical value c_{v3} of the converted single layer is used to calculate the required time for consolidation t by Formula (3.6.7).

$$t = \frac{(H_0/2)^2}{c_{V0}} Tv \quad \cdots\cdots (3.6.7)$$

Where

T_v : time factor (a selected value from Fig. 3-6-5 in correspondence with the mean degree of consolidation can be used)

c_{v0} : typical c_v of the consolidation layer (c_{v3} in the case of Formula 3.6.7)

When the total consolidation settlement is S_f in the case where only one-dimensional consolidation settlement below the central part of an embankment is considered, the settlement S_c when the degree of consolidation reaches the point of U and S_f have the relationship expressed as $S_c = S_f \cdot U$.

Consequently, the relationship between the time t and the settlement S_c, i.e. the settling speed, can be obtained following the above procedure.

In the case of a soil layer consisting of organic soil (particularly peat), special attention is required to the fact that the settling speed obtained by Formula (3.6.7) using the value of c_v given by the standard consolidation test is often much slower than the actual settling speed.

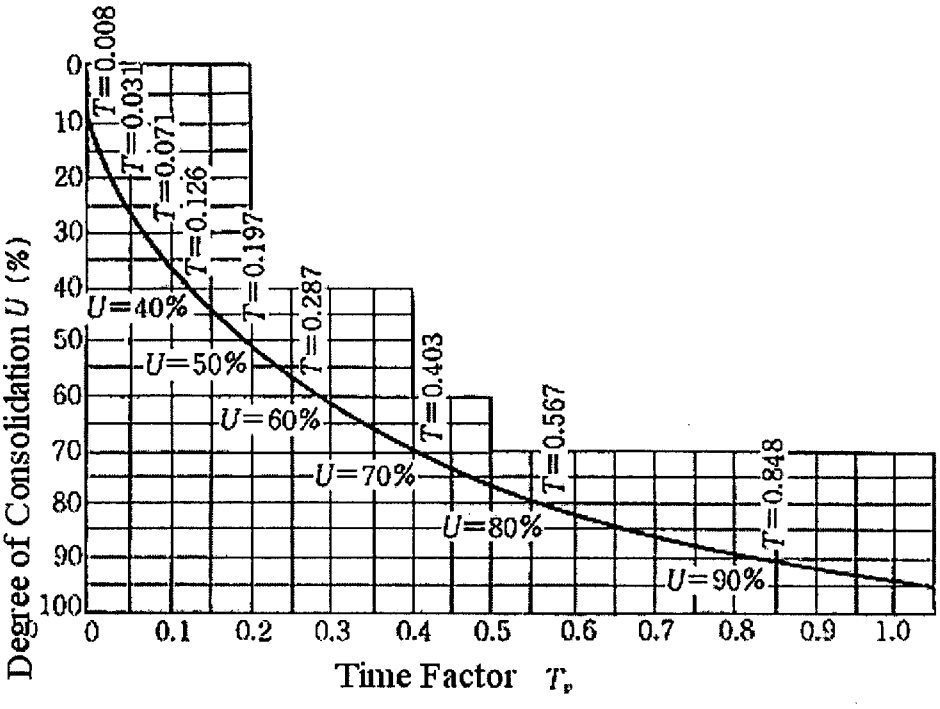


Fig. 3-6-5 Relationship between U and T_v

(3) Residual Settlement

Residual settlement means that settlement which is expected to continue after a reference point of time. In the case of a road embankment, the time when paving work is completed or the time when a newly constructed road serves traffic for the first time constitute such a reference point of time.

At the design stage, the difference between the total settlement S, consisting of consolidation settlement and immediate settlement, and the settlement S_t from the commencement of banking work to the reference point of time t as shown in Fig. 3-6-6 is used to express the residual settlement ΔS_t in the following manner.

$$\Delta S_t = S - S_t \quad \text{..... (3.6.8)}$$

Meanwhile, the residual settlement ΔS_t can be calculated using Formula (3.6.9) based on the one-dimensional consolidation.

$$\Delta S_t = S_c(1-U) \quad \text{.....(3.6.9)}$$

Where,

S_c : settlement by one-dimensional consolidation

U : degree of consolidation at the reference point of time

However, it is known that secondary consolidation settlement continues for a long period of time in the case of real ground. It is, therefore, necessary to take such prolonged consolidation settlement into consideration although it is difficult to accurately estimate the total settlement, including secondary consolidation. The reality is that the time-settlement curve based on the one-dimensional consolidation theory is still not very accurate.

Given the above situation, the consolidation settlement ΔS, which occurs from the reference point of time t to a later time with the passed time of Δt, is used to express the residual settlement.

The residual settlement at an ordinary banking section is calculated based on Δt of three years and the reference point of time being the completion time of the paving work. Such reference point of time and the value of Δt vary depending on the subject of analysis, such as a culvert or over-raise of the subgrade, etc. When the residual settlement ΔS obtained by the above detailed calculation of settlement exceeds the design target value, a possible change of the work schedule and the necessary for a countermeasure(s) should be examined.

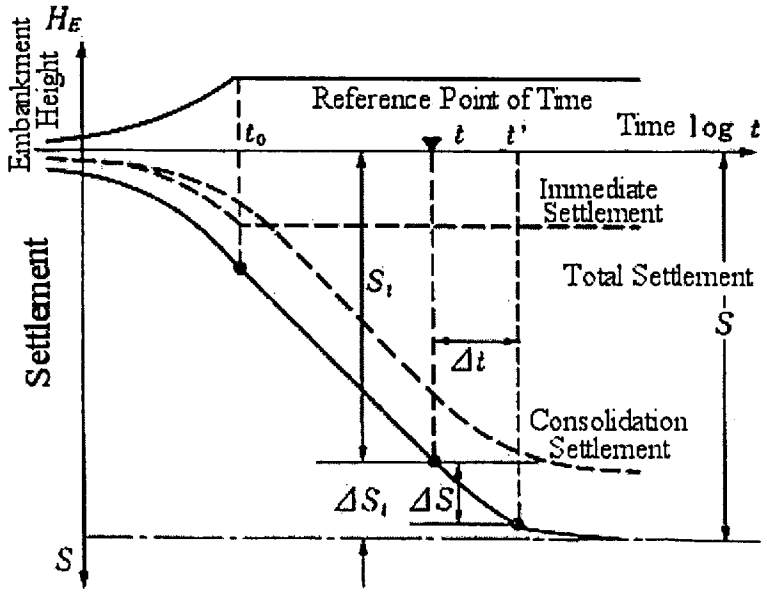
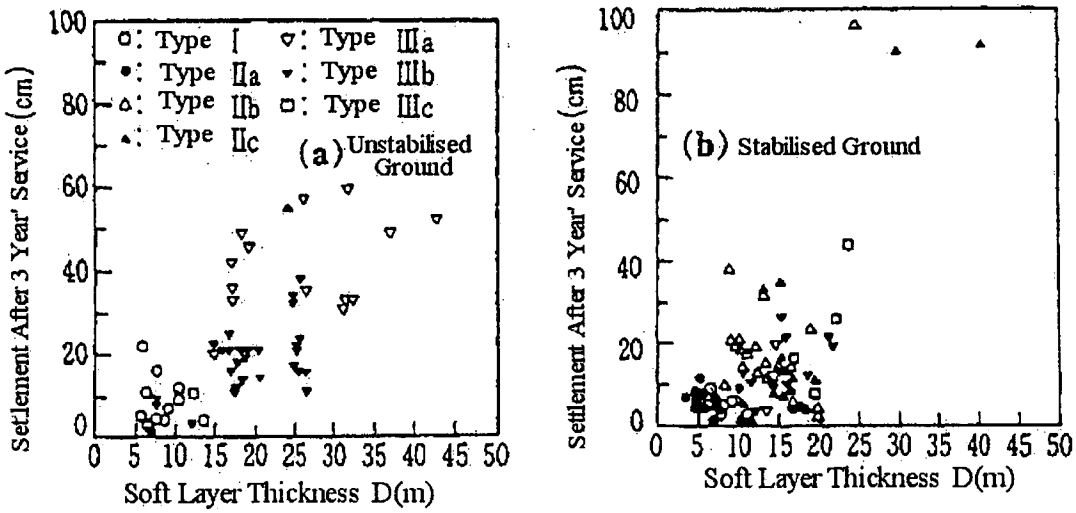


Fig. 3-6-6 Residual Settlement

The target allowable residual settlement at the central part of an embankment for a three year period after the completion of paving work should preferably be established using the following values as yardsticks and taking the importance of the road in question into consideration.

- Banking section connected to a bridge or viaduct : 10 – 30 cm
- Other sections: to be determined by analysing the effects of settlement

Fig. 3-6-7 shows the relationship between the settlement of an expressway after three years of service and the soft layer. According to this figure, the settlement after three years of service is approximately 20 cm for a soft layer thickness of up to 15 m regardless of the ground type. The settlement tends to be large in the case of a deep layer type where the thickness of the soft layer is approximately 15 m or thicker. Road surface repair work necessitated by soft ground is often conducted after several years of service. Except in the case where the soft layer thickness is large, the burden of repair work can be reduced by opting for a long period during which the embankment is left as it is before the open to traffic.



(a) Without Stabilisation Work (b) With Stabilisation Work

Note

Type I (shallow layer type)	: shallow ground with a soft layer thickness of approximately 10 m
Type II (peat type)	: ground mainly consisting of peat
Type IIa (peat along type)	: ground where peat alone is deposited immediately below the surface layer
Type IIb (peat and clay type)	: deposit of clay under peak with clay mixed with organic sand or with sand in-between
Type IIc (peat insertion type)	: deposit of peak below a 2 - 3 m thick cohesive soil surface
Type III (deep layer type)	: deposit ofthick marine or terrestrial clay deep below a soft soil layer of more than 10 m in thickness
Type IIIa(upper sand layer type)	: deposit of thick marine clay below a thick sand layer, in turn located below a virtually non-existent or thin (1 - 2 m) upper clay layer
Type IIIb (sand layer insertion type)	: deposit of thick marine clay below a relatively thick upper clay layer (4 - 5 m) with an intermediate sand layer in-between
Type IIIc (continual type)	:deposit of marine layer immediately below surface clay (clay containing organic matters in most cases) without an intermediate sand layer

Fig. 3-6-7 Settlement after 3 Years' Service and Classification of Soft Layer Thickness

3.6.4 Stability Analysis for Embankment on Soft Ground

(1) Limit Embankment Height

The stability of an embankment on soft ground can usually be assessed by stability analysis and other means. At the preliminary examination stage, it is possible to use the relationship shown in Fig. 3-6-8 to calculate the ultimate bearing capacity q_d of the ground to the mean unconfined compressive strength q_u of the soft layer so that the limit embankment height H_{Ec} can be calculated using Formula (3.6.10).

$$H_{Ec} = q_d / \gamma_E \quad \cdots\cdots (3.6.10)$$

Where,

γ_E : unit volume weight of embankment

To use Fig. 3-6-8, the required value can be selected from Table 3-6-4 depending on the soft layer thickness and the soil type.

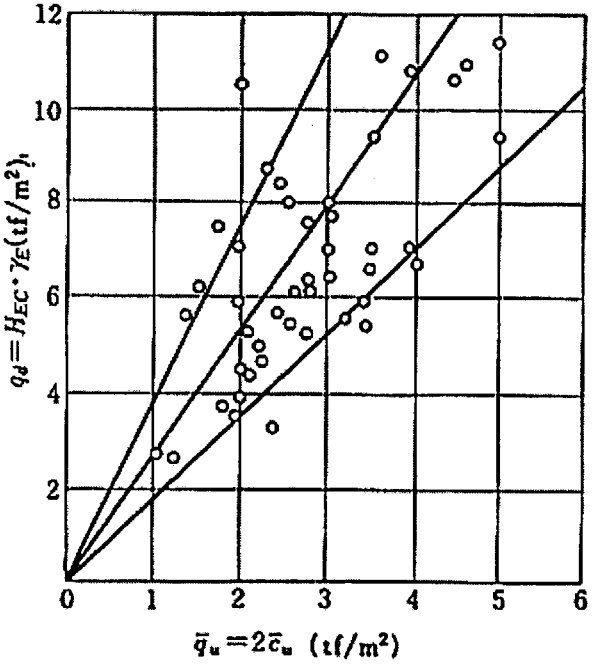


Fig. 3-6-8 Relationship between q_u and Limit Bearing Capacity q_d

Table 3-6-4 Limit Bearing Capacity

Ground Conditions	Limit Bearing Capacity q_d (tf/m²)
Thick clayey ground and muck or peaty ground with thick deposit of organic soil	3.6 $\bar{c}_u$
Ordinary clayey ground	5.1 $\bar{c}_u$
Thin clayey ground and muck or thin peaty ground with little organic soil	7.3 $\bar{c}_u$

(2) Stability Analysis for Embankment

Stability analysis against sliding failure of the foundation ground, including the embankment, is generally conducted to check the stability of a planned embankment on soft ground and the impacts of such an embankment on adjacent structures. Analysis of the stability against sliding failure normally uses the circular sliding calculation method and particular attention should be paid to the points described below in such calculation. For this purpose, the minimum safety factor should always be 1.2 or higher.

1) Shape of Sliding Surface

The assumed sliding surface is that of which the value of q_u is small (as $\phi_u = 0$) and of which the passing length in the layer of slow consolidation because of its thickness and its c_v value is the longest. In reality, the shape of the sliding surface tends to be a very complicated curved surface but the assumption that the sliding surface is circular does not create many practical problems.

2) Sliding Surface inside Embankment

Calculation considering the shear resistance along the assumed sliding surface is commonly conducted in regard to sliding inside an embankment. However, in the case of an embankment above thick soft ground, it is likely that the embankment faces tensile force due to the lateral movement of the ground. For rough stability analysis, the shear resistance inside the embankment can be ignored and the calculation can be conducted based on the assumption that vertical cracks along the entire height of the embankment occur (Fig. 3-6-9).

When the load gradually increases over a long period of time, the above method produces a smaller safety factor than the actually required safety factor. In this case, it is necessary to conduct calculation using Formula (3.6.11) which takes the increased strength due to consolidation of the foundation ground into consideration (Fig. 3-6-10).

$$c_u = c_{u0} + m(p_o - p_c' + \Delta p) \cdot U \quad \text{..... (3.6.11)}$$

However, $p_o + \Delta p \le p_c'$; $c_u = c_{u0}$

$p_0 + \Delta p > p_c'$; $c_u = m \cdot p_t = m\{p_c' + (p_0 - p_c' + \Delta p) \cdot U\} = c_{u0} + m(p_0 - p_c' + \Delta p) \cdot U$

Where,

c_{u0} : undrained cohesion of soil at the original ground prior to banking (tf/m^2)

m : rate of strength increase

p_0 : overburden pressure prior to banking (tf/m^2)

p_c' : $p_c' = c_{u0} / m$ (tf/m^2)

Δp : increased stress to soil on the sliding surface caused by the embankment load (tf/m^2)

U : degree of consolidation of the soil on the sliding surface

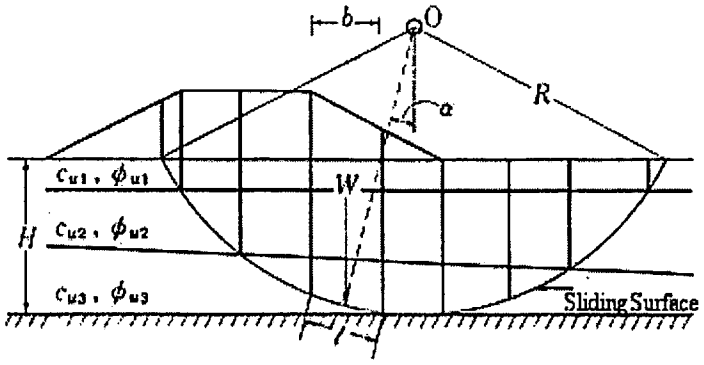


Fig. 3-6-9 Stability Analysis by Slice Method

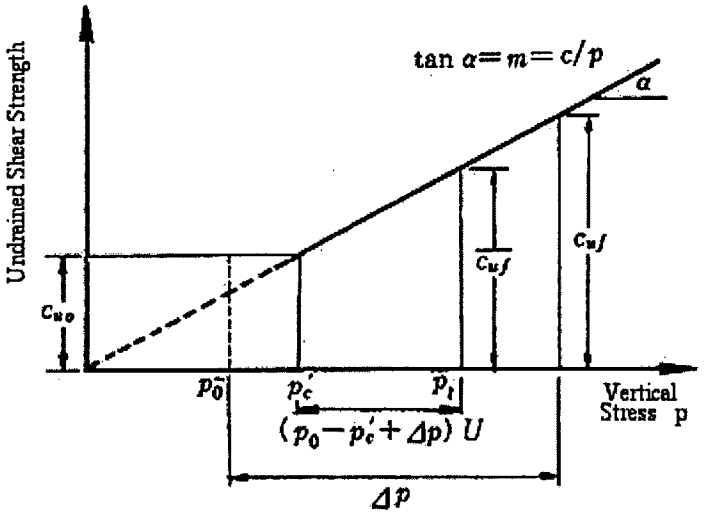


Fig. 3-6-10 Shearing Strength Considering Increased Strength by Consolidation

3.6.5 Supplementary Matters for Analysis of Settlement and Stability

(1) Means and Timing of Estimating Settlement

As settlement calculation involves many assumptions and uncertain elements, it is rational to treat the calculated settlement value as a yardstick with corrections being made to this predicted value based on field observation at the construction and maintenance stages.

(2) Impact on Side Ground and Its Scope

Prior prediction of the changing form and scope of the side ground which experiences upheaval during the construction period of an embankment and the subsequent settlement lasting for a long period of time after the completion of an embankment and also of the impact and scope of sliding failure of an embankment on

the side ground is extremely important from the viewpoints of preventing harm by construction work and ensuring environmental conservation.

Fig. 3-6-11 shows the degree and scope of impact on the surrounding ground of embankments which have been constructed in Japan above soft ground.

According to Fig. 3-6-11, the upheaval of the side ground at the time of embankment failure is not particularly large. However, the figure shows that the scope of impact can become extremely wide with increases of the ratio between the base width of the embankment and the thickness of the soft layer. Careful attention is particularly required when the basement layer of the soft layer is dipped as landslide-like failure along the direction of inclination is believed to cause damage over an extremely large area.

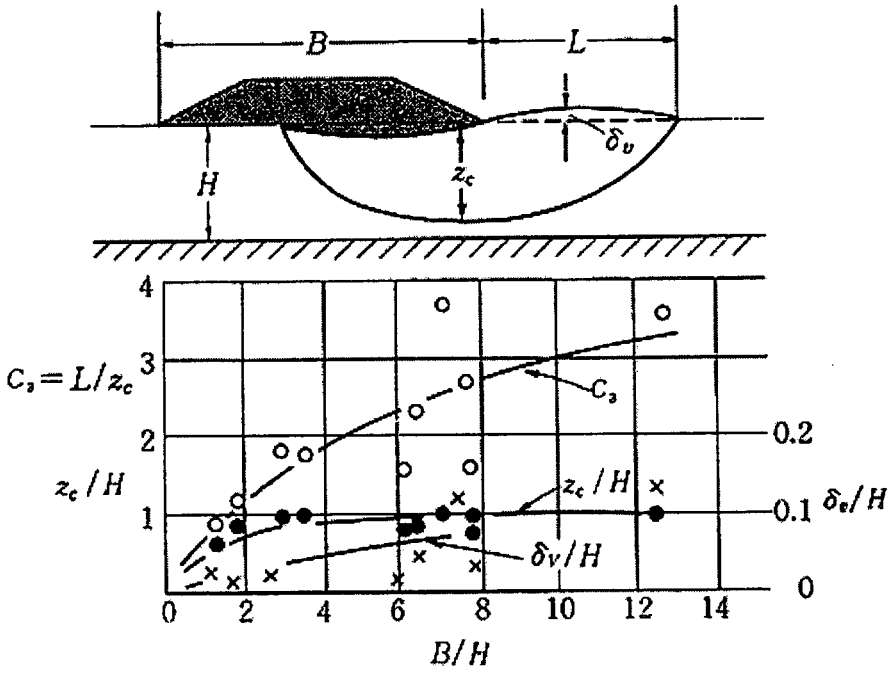


Fig. 3-6-11 Impacts on Surrounding Ground

(3) Settlement of Low Embankment Road

When a so-called low embankment road, i.e. a road on an embankment of 2 – 3 m in height, is constructed on soft ground, the traffic load cannot be sufficiently dispersed inside the embankment. As a result, problems often occur in terms of maintenance of the flatness of the road surface and maintenance of the pavement.

Fig. 3-6-12 shows the converted values of the impacts of traffic load corresponding to the embankment thickness to the embankment load based on the settlement curve established by field observation of low embankment roads. To be more precise, for low embankment roads which are subject to a high traffic load

after opening and for which large settlement is predicted, the settlement after opening and the size of the surcharge to deal with such settlement can be examined using the embankment load corresponding to the impacts of the traffic load obtained from this figure.

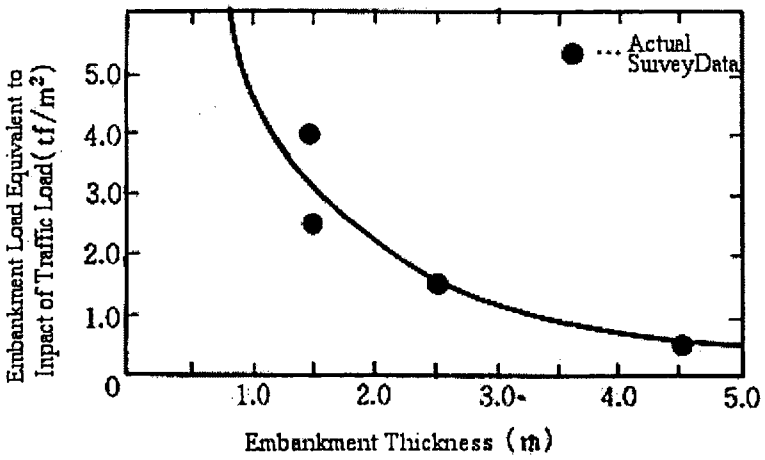


Fig. 3-6-12 Embankment Load Equivalent to Impact of Traffic Load

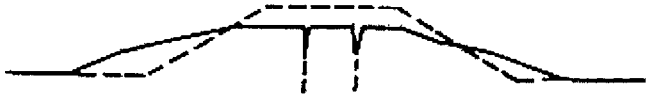
3.6.6 Analysis for Earthquake

It is a common occurrence for the stability of an embankment and other structures constructed above soft ground or on inclined ground to temporarily decline due to an earthquake, resulting in such damage as cracks, settlement and sliding failure. As shown in Fig. 3-6-13, an earthquake can inflict various types of damage of different scales to road embankments, including longitudinal as well as cross-sectional cracks, faulting, cave-in and failure, depending on the conditions relating to a specific embankment, ground and earthquake motion. Past experience of damage indicates that the type of damage shown in Fig. 3-6-13(-a) tends to occur with clayey ground while the type of damage shown in Fig. 3-6-13-(b) tends to occur as a result of the large-scale liquefaction of sandy ground. Meanwhile, the type of damage shown in Fig. 3-6-13-(c) tends to occur with peaty ground.

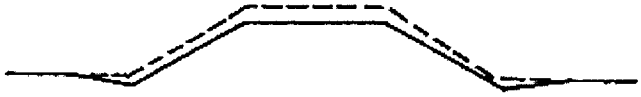
It is, therefore, desirable to conduct appropriate analysis of and consideration of earthquakes, if required, in regard to those road sections where earthquake damage will have a significant adverse impact on road traffic because of the difficulty of restoration or where an earthquake will cause secondary damage to adjacent important structures. To be more precise, it is recommended that analysis be conducted based on the assumption that an embankment will be damaged by an earthquake and to prepare and apply necessary measures when the foundation ground is a loose sandy soil layer of which the liquefaction at the time of an earthquake is anticipated (see 2.5.3 – Survey for Judgement on Liquefaction) or a particularly soft clayey soil layer which may experience significant disturbance at the time of an earthquake.



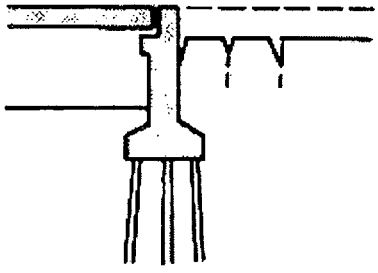
(a) Failure of embankment



(b) Failure of the ground



(c) Sttlement of embankment



(d) Sttlement of embankment around the approach to structure

Fig. 3-6-13 Types of Damage to Embankment (Case of Soft Ground)

Stability analysis for an embankment above ground which is prone to liquefaction usually involves calculation of the stability against circular sliding using the seismic coefficient method. The timing of analysis, external forces to be considered and the manner of handling the strength differ depending on the scale of the target earthquake (see Table 3-6-5). In the case of medium-scale earthquake motion, the ground stability is analysed using a formula which ignores the inertia force at the time of an earthquake while taking the impact of excess pore water pressure generated by earthquake motion as indicated by Formula (3.3.4) into consideration. The minimum safety factor for medium-scale earthquake motion is not less than 1.0.

For improvement of the stability of an embankment above ground which is prone to liquefaction, prevention of the liquefaction of the ground is a more direct measure than attempting to increase the strength of the embankment itself. To prevent liquefaction, the application of such countermeasures as an increase of the ground density,

increase of the confining pressure, dissipation of excess pore water pressure, solidification and the suppression of shear deformation is desirable.

Table 3-6-5 Evaluation Method of Stability of Embankment above Ground Liable to Liquefaction

Scale of Earthquake Motion	Point of Analysis	Method	External Force	Strength
Medium-Scale Earthquake Motion (L-1)	During and after earthquake	Effective stress analysis	Gravity	Static strength, taking excess pore water pressure into consideration
Large-Scale Earthquake Motion (L-2)	During earthquake	Total stress analysis	Gravity; earthquake intensity	Dynamic strength, taking initial shear stress into consideration
	After earthquake	Total stress analysis	Gravity	Residual strength after liquefaction

3.6.7 Countermeasures for Soft Ground

(1) Types and Effects of Various Countermeasures

There are many types of countermeasures to stabilise soft ground which have different characteristics and intended effects.

The purposes of countermeasures for soft ground are classified into those for settlement, those for stability and those for earthquakes and their effects are categorised as shown in Table 3-6-6.

Table 3-6-6 Purposes and Effects of Countermeasures for Soft Ground

Purposes	Effects	Category
Countermeasures for Settlement	Facilitation of consolidation settlement: to facilitate the settlement of the ground to reduce the harmful residual settlement	A
	Reduction of total settlement: to reduce ground settlement itself	B
Countermeasures for Failure	Suppression of shearing deformation: to suppress the upheaval or lateral movement of the ground around an embankment caused by the embankment load	C
	Reduction of embankment load: to ensure stability by reducing the embankment load	D
	Facilitation of strength increase: to ensure stability by increasing the ground strength	E
	Increase of sliding resistance: to increase the sliding resistance to ensure stability by changing the form of an embankment or replacing part of the ground soil	F
Countermeasures for Earthquakes	Prevention of liquefaction: to prevent liquefaction to ensure stability at the time of an earthquake	G

Table 3-6-7 Types and Effects of Countermeasures for Soft Ground

Countermeasures		Description	Effect
Surface soil stabilization	▪ Basal reinforcement method ▪ Shallow soil stabilization method ▪ Surface drainage method ▪ Sand mat method	Mechanical work for soft soil stabilisation and banking is made easier by spreading a geotextile polymer cloth, net, wire mesh or fascine on the surface of the foundation ground, treating the said surface with lime or cement or improving the said surface through the introduction of drainage ditches. In the case of the sand mat method, this differs from other methods in that it forms a consolidated drainage layer and is commonly used in conjunction with vertical drain or other types of work designed to promote consolidated drainage.	D E F
Replacement of ground	▪ Replacement method by excavation ▪ Forced replacement method	This type of work involves the replacement of part or the whole of soft soil with good quality soil. Soil replacement gives shear resistance, increases the safety factor and reduces settlement in proportion to the scale of soil replacement. Two methods are available based on the manner of soil replacement, i.e. either excavation or forced replacement using the weight of the embankment. Soil replacement is sometimes conducted using crushed stone which is unlikely to suffer from liquefaction to prevent liquefaction due to an earthquake.	B C G
Counterweight embankment	▪ Counterweight filling method ▪ Gentle slope method	This type of work is designed to prevent the sliding failure of an embankment by increasing the moment to resist sliding, in turn by applying counterweight fill to the side faces of the embankment or adopting a gentler slope gradient. As the height of the side faces of the embankment is not suddenly increased, the lateral flow remains small. Sometimes, the counterweight fill is removed after an increase of the strength due to consolidation.	C
Reinforced embankment	▪ Reinforced embankment method	This type of work is designed to prevent the lateral flow and sliding failure of the ground with abrasion between the reinforcing materials and soil and the pull-out resistance force of such materials by placing reinforcement materials, such as steel netting, anchor reinforcing bars with a band steel plate or geotextile to the embankment.	C
Load reduction	▪ Lightweight embankment method	This method is designed to reduce the adverse impact of an embankment on the original ground by reducing the weight of the embankment body. For this purpose, expanded polystyrol, expanded mortar, pumice, quenched blast furnace slag and/or coal ash are used as the banking materials. In addition, hollow structures, such as corrugated pipes and culverts, may be placed inside the embankment to reduce the nominal weight of the embankment.	
Slow banking	▪ Gradual loading method ▪ Stage loading method	This type of work involves the slow construction of an embankment. As a strength increase due to consolidation is expected to take place, an embankment can be safely constructed at a place where quick banking work cannot ensure its stability. These two methods represent different ways of loading, i.e. the gradual but steady construction of an embankment and the temporary suspension of banking work from time to time to allow an increase of the ground strength. This type of work is often employed in conjunction with the vertical drain and other methods	C

Surcharge	▪ Surcharge method by banking ▪ Vacuum consolidation method ▪ Groundwater lowering method	This type of work is designed to reduce the settlement of a structure by constructing a structure only after the forced settlement of the ground by applying a load to the ground on which the construction of an embankment or structure is planned. Surcharge normally takes the form of the embankment but some methods use the effective stress increased by water or air pressure or reduction of groundwater through well points.	E
Vertical drain	▪ Sand drain method ▪ Cardboard drain method	This type of work is designed to facilitate consolidation settlement and also to increase the strength of the ground by shortening the lateral distance for consolidation drainage through the vertical insertion of sand columns or cardboard at suitable intervals in the ground. Gravel or crushed stone may be used instead of sand. There are many methods, including the use of bags or casing, to wrap the sand columns and ropes may be used instead of cardboard. The actual work is conducted in many ways, including the driving of steel pipes or the insertion of steel pipes using vibration to create sand columns and boring of the ground using a water jet to insert the sand columns.	C E
Sand compaction pile	▪ Sand compaction pile method	This type of work is designed to reduce settlement by increasing the stability with the bearing capacity of compacted sand piles in the ground. These sand piles also compact into a soft layer. There are many ways to create sand compaction piles, including hammering and vibration. Crushed stone may be used instead of sand.	A C
Vibro-compaction	▪ Vibro-floatation method	A bar-shaped vibrator is inserted into loose sandy ground and water is added to the vibrating section to compact the ground with the effects of vibration and water. Sand or gravel is injected near the vibrating section to form a sand pile to improve a loose sandy soil layer to a compacted sandy soil layer.	B F
	▪ Rod compaction method	This method has been developed to compact loose sandy ground. A bar-shaped (rod) vibrator inserted in the ground is given vertical vibration and is gradually pulled out while compacting the ground. As this method compacts the ground with vertical vibration, it can effectively use the weight of the soil.	B F
	▪ Falling weight compaction method	A weight is dropped onto the ground to compact the ground while the resulting excess water is drained to increase the shear strength. As vibration and noise are produced, advance examination of the environmental and working conditions is required. One advantage is that the improvement effects can be checked immediately after the completion of the work.	B C
Soil solidification	▪ Deep mixed method of soil stabilisation	This method aims at preventing settlement and sliding failure by increasing the strength of the ground through the columnar or total stabilisation of the ground with a stabilisation mixture, such as cement or lime and soil of the original ground, for a certain depth from the surface of soft ground. The construction machine use is either an agitating blade type or injection type.	C
	▪ Lime stabilisation method	This method aims at increasing the stability and reducing the settlement of the ground by increasing the strength of the ground through the consolidation of the ground with water absorption and chemical binding by quick lime columns created in the ground.	

	▪ Chemical grouting method	This method aims at reducing the permeability and increasing the strength of the original ground by grouting chemicals into the ground	
	▪ Freezing method	This method aims at artificially freezing pore water in the ground by means of installing steep piles called freezing pipes into the ground. This may be conducted as temporary works.	
Structural method	▪ Sheet piling method	This method aims at increasing the stability by reducing the lateral deformation of the ground through the insertion of sheet piles at the sides of an embankment. It also reduces upheaval and the adverse effects of settlement on the surrounding ground.	
	▪ Pile works	Ready made piles are driven into the ground to achieve a similar effect to that of the sheeting method. Piles are driven into the lower part of an embankment to increase the stability of the embankment body and to reduce settlement. The slab method involves the introduction of a slab above the piles to enhance the supporting function of the piles and an embankment is constructed above the slab. This method is sometimes used for an embankment behind an abutment. The pile heads may be linked with reinforcing bars before banking.	
	▪ Culvert (elevated structure)	When an embankment is perceived to be unstable due to a large load, possibly resulting in large settlement, a culvert may be planned instead of an embankment. Another method involves a line of corrugated pipes instead of a culvert as the base for banking. An elevated structure may be opted for when a countermeasure for soft ground involving banking work is very expensive or causes concern in regard to its future maintenance.	

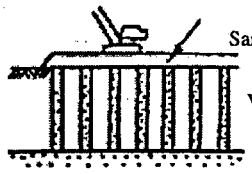
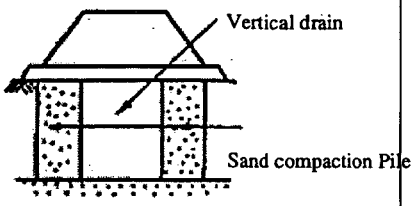
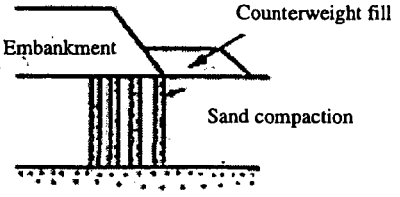
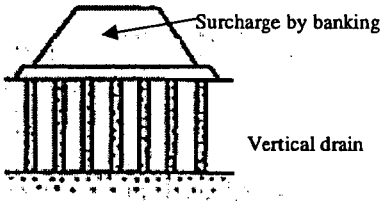
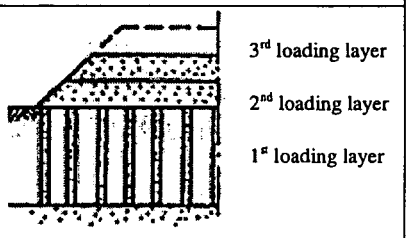
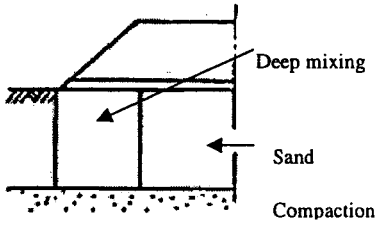
Note) ○: most dominant effect

While the effects of various types of countermeasures differ from each other, it is often the case that a countermeasure has a main effect as well as incidental secondary effects. Table 3-6-7 shows the effects of countermeasures for soft ground using the symbols used in Table 3-6-6. The main category of effect expected of each type of countermeasure is indicated by bracketing of the category symbol to distinguish it from any secondary effects.

(2) Selection of Countermeasures

For the selection of an appropriate countermeasure(s), proper examination of the reason and purpose of introducing a countermeasure is essential, followed by consideration of the various conditions, including properties of the ground, road structure, working conditions and possible impacts on the surrounding area so that the optimal as well as most economic countermeasure suiting the required purpose is selected. Although the countermeasures in question can be applied individually, they tend to be applied in combination as shown in Table 3-6-8.

Table 3-6-8 Examples of Combined Countermeasures

Countermeasures for Settlement	Stabilisation Measures		Illustrations of Examples
Vertical drain method	To secure trafficability	Surface soil stabilisation (ex. sand mat method)	
Vertical drain method	To stabilise structure	Sand compaction pile method	
Surcharge method by banking		Counterweight fill method or sand compaction pile method	
Surcharge method by banking and vertical drain method			
Vertical drain method		Slow loading method	
Sand compaction pile method		Soil Solidification (ex. deep mixing method for soil stabilisation)	

(3) Application of Countermeasures

Frequently encountered problems in the process of dealing with soft ground are described below together with a description of the appropriate countermeasures to be applied.

1) Low Embankments

The problems of low embankments above soft ground are already discussed in 3.6.5-(3) – Settlement of Low Embankment Roads. There are basically three countermeasures as listed below to deal with the identified problems.

① Creation of a highly rigid embankment structure
② Good drainage inside the embankment
③ Reduction of settlement near the surface layer of the foundation ground and uniformatisation as well as increase of the ground strength

The countermeasure to achieve ① and ② is the selection of good quality soil and full compaction. Meanwhile, the following countermeasures are believed to be effective to achieve ③.

- Surface soil stabilisation work (sand mat method, basal reinforcement method, shallow soil stabilisation method and surface drainage method)
- Surcharge method
- Replacement method (see Fig. 3-6-14)
- Sand compaction pile method and soil-solidification method

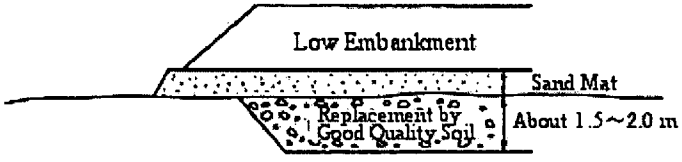


Fig. 3-6-14 Replacement Method

2) Half-bank and Half-cut Section

A half-bank and half-cut section here means that soft ground forms the foundation for cross-sectional part of an embankment as shown in Fig. 3-6-15. In the case of this type of embankment, damage to the pavement often occurs due to cracks in the road surface, in turn caused by sliding failure and differential settlement along the connection face of the ground. This sliding failure and differential settlement are the result of the settlement of the soft ground. As the thickness of the soft ground at these sites generally increases towards the direction of a valley, not only differential settlements in the embankment but also sliding of the

embankment towards the valley tend to occur. To combat such phenomena, the following countermeasures are often employed.

① Improved connection at the boundary between the embankment and the ground (see Fig. 3-6-15-(a)

② Reduction of the total settlement of the soft ground to increase the strength of the ground

- Case of thick soft layer: sand compaction pile method and soil-solidification method, etc. (see Fig. 3-6-15-(b))
- Case of thin soft layer: replacement method and shallow soil stabilisation method (see Fig. 3-6-15-(c))

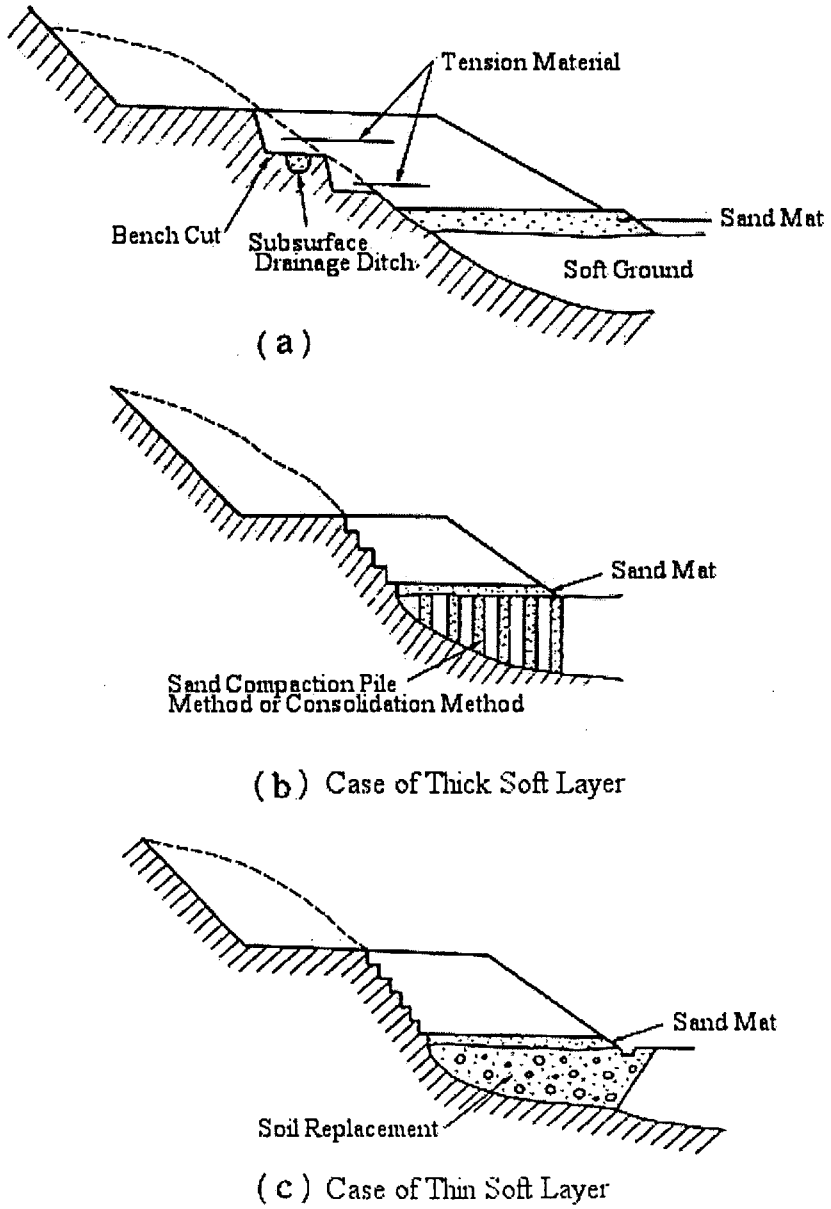
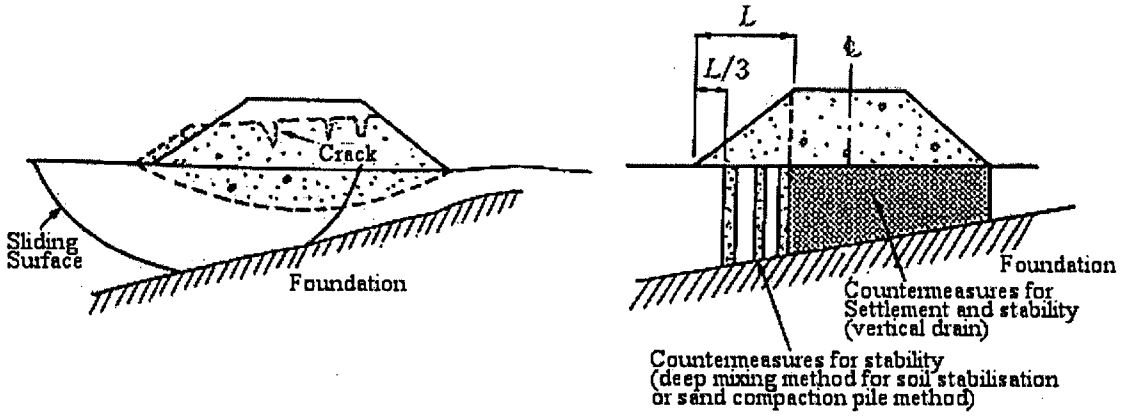


Fig. 3-6-15 Countermeasures at Half-Bank and Half Cut Section

3) Embankment on Inclined Ground

In the case of an embankment on an inclined ground, the settlement is larger at the thicker section of the soft layer as shown in Fig. 3-6-16-(a), creating a risk of the occurrence of sliding towards the said thicker soft soil section.

Given the fact that differential settlement facilitates sliding failure, what is required is reduction of the settlement as much as possible. From this viewpoint, the sand compaction pile method and the deep mixing method for soil stabilisation, etc. should prove effective (Fig. 3-6-16-(b)). When a countermeasure such as these is to be employed, it is a good idea to make the settlement even by adopting a shorter interval for the piles, etc. for the thick soft soil section and a longer interval for the thin soft layer section.



(a) Sliding Failure Along Foundation

(b) Combination Example of Countermeasures

Fig. 3-6-16 Sliding Failure of Inclined Ground and Examples of Countermeasures

4) Widening of Embankment

In Fig. 3-6-17, existing embankment A is stabilised because of the completion of settlement at the level of settlement S_1 but the widening of embankment B causes fresh settlement of embankment A towards the embankment B side, causing unevenness of the road surface on embankment A and resulting in cracks in the pavement. Some concrete examples of countermeasures to eliminate such an adverse impact are listed below.

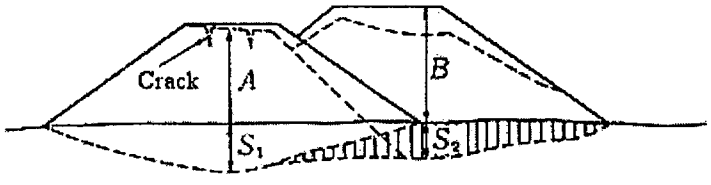


Fig. 3-6-17 Deformation of Existing Embankment by Widening of Embankment

① Sheet piling method (see Fig. 3-6-18)

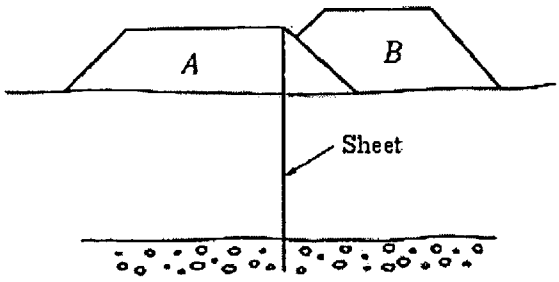


Fig. 3-6-18 Sheeting Work

② Sand compaction pile method or soil-solidification method, etc.

③ Pile net (see Fig. 3-6-19)

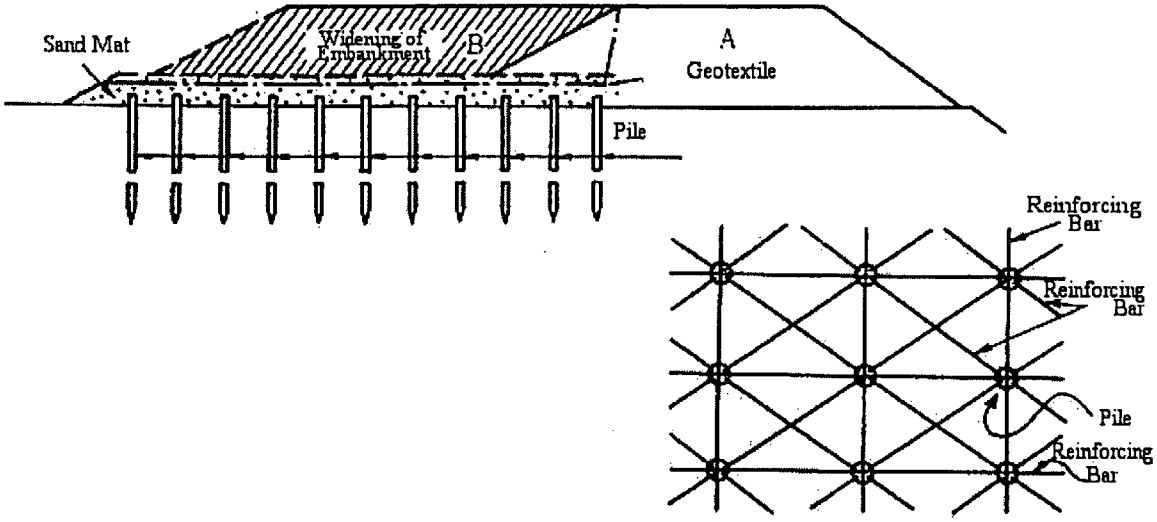


Fig. 3-6-19 Example of Settlement Control Work using Pile Net

5) Work to Prevent Adverse Impacts on Surrounding Ground

When important facilities or houses are located adjacent to the planned embankment, it is necessary to prevent any adverse impacts due to lateral flow or settlement of the embankment on these structures. While the types of work (countermeasures) to meet such a necessity are often those described in 4) – Widening of Embankment, some types of work causing vibration and/or noise may not be applicable.

In some cases, construction methods with relatively little vibration or sound, such as the deep mixing method for soil stabilisation and the lime pile method, may be used at the bottom of the slope of an embankment as shown in Fig. 3-6-20.

6) Embankment in Contact with Structure

When an embankment above soft ground is in contact with such structures as an abutment, retaining wall or other, the following problems occur (see Fig. 3-6-21).

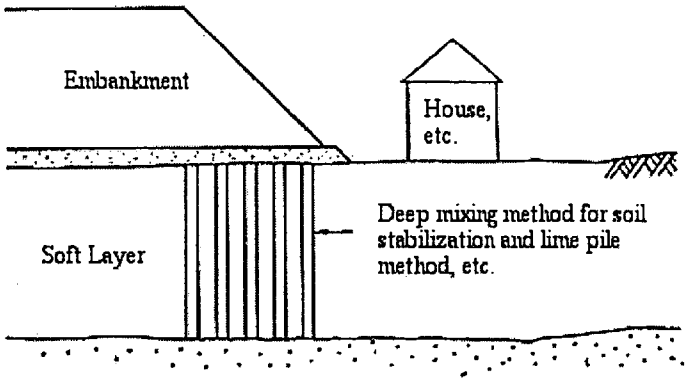
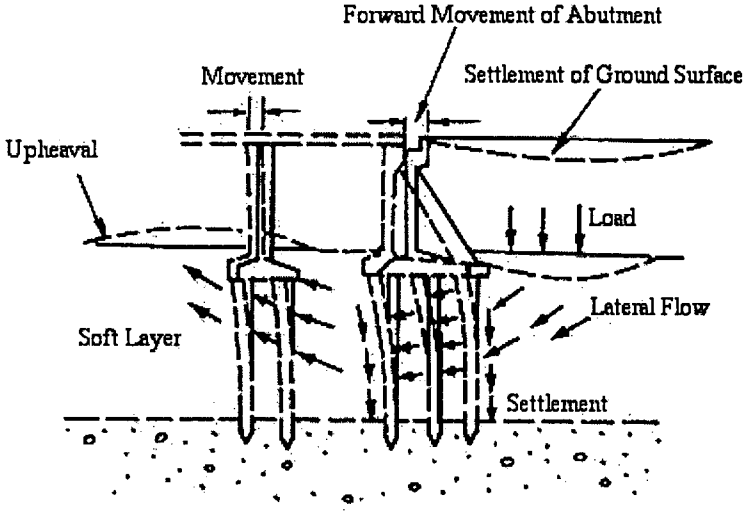
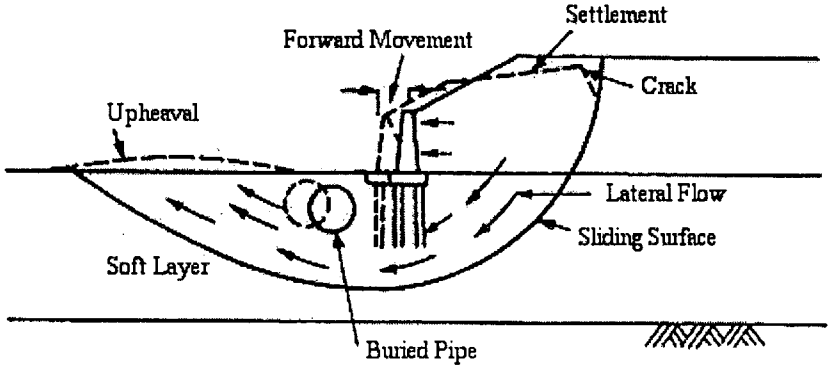


Fig. 3-6-20 Prevention Method for Adverse Impacts on Neighbouring Ground of Embankment



(a) Forward Displacement of Abutment and Pier and Differential Settlement of Road Surface



(b) Displacement of Retaining Wall and Buried Pipe, etc.

Fig. 3-6-21 Displacement of Structure and Differential Settlement of Road Surface Adjacent to Embankment

- The settlement and lateral flow of the ground occur due to the surcharge of the embankment at the back of the structure. As a result, differential settlement occurs at the approach of the embankment to the structure.

- The settlement and lateral flow of the ground cause displacement of the structure while lateral flow and negative friction occur with the foundation

Given the above prospect of negative development, banking work must follow the processes designed to contain settlement and with little impact on structures. Any countermeasure to be selected must be capable of reducing differential settlement and containing the settlement and lateral flow of the ground.

Some application examples of such countermeasures are described next.

① Surcharge Method (see Fig. 3-6-22)

Firstly, a surcharge embankment is constructed to facilitate settlement to strengthen the ground. This surcharge embankment is then removed by means of excavation to build a structure, such as an abutment, on the strengthened ground. In this case, vertical drain method is often simultaneously conducted at the foundation to enhance the effect of the surcharge work.



Fig. 3-6-22 Example of Surcharge Method

② Soil-Solidification Method (see Fig. 3-6-23)

In some cases, surcharge work may not be feasible because of a limited construction period or the site conditions. In such a case, soil-solidification method is conducted. There are two types of soil improvement by soil solidification based on the improvement depth. One is improvement of the entire soft layer and the other is the floating type to improve part of the soft layer.

③ Culvert Method and Light Weight Embankment Method

Culvert work is designed to use culverts at the banking section behind an abutment to reduce the embankment load so that the lateral flow toward the abutment is reduced together with a reduction of the settlement of the approach. Instead of using a culvert, a light weight banking material may be used as the back-filling material at the back of the embankment.

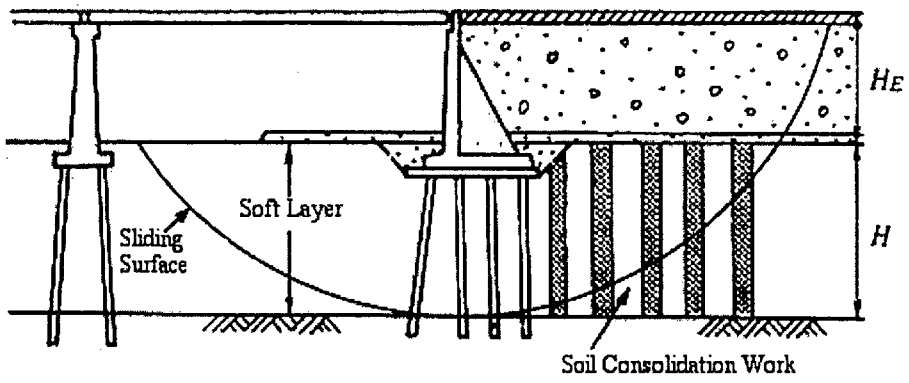


Fig. 3-6-23 Example of Soil Solidification Work

④ Counterweight Embankment Method

As shown in Fig. 3-6-24, counterweight embankment is applied to the front of an abutment for the purpose of creating resistance to the embankment at the back. An earth pressure-relieved abutment adopts the same idea in that a small abutment supported by footings is introduced to the embankment at the front in order to reduce the earth pressure on this small abutment.

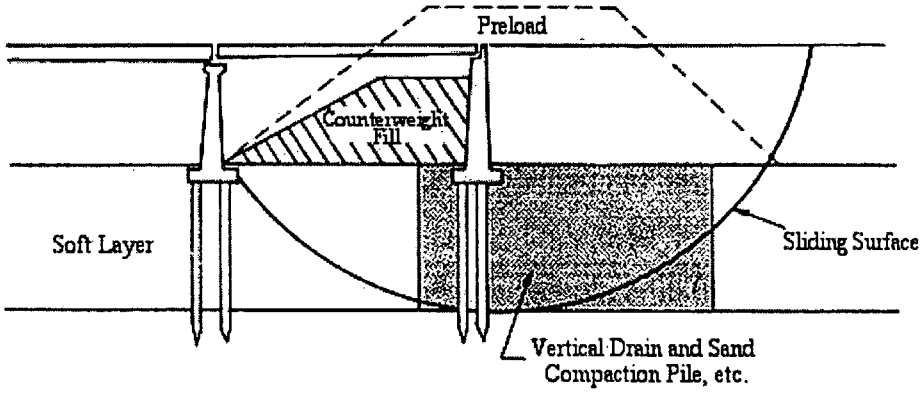


Fig. 3-6-24 Example of Counterweight Embankment Method

⑤ Pile Work

Pile work involves the insertion of piles in the foundation ground for an embankment and the placing of a concrete slab above the pile heads so that the load of the embankment at the back can be conveyed to the bearing ground via the piles to contain the settlement of the road surface and to simultaneously reduce the lateral flow of the ground by the lateral resistance of the piles.

7) Culvert

There are two ways of introducing a culvert above soft ground. One is the use of bearing piles to prevent settlement and the other is the use of spread foundations to allow settlement to a certain extent.

Lateral flow and settlement are deal with by the following countermeasures.

① Surcharge method

When a high level of residual settlement is anticipated, a camber corresponding to the anticipated residual settlement is introduced along the longitudinal axis of the culvert so that the design height is achieved with the completion of residual settlement.

② Sand compaction pile method or soil-solidification method

③ Friction pile method

④ Replacement method

8) Countermeasures for Earthquakes

The basic requirements to prevent or to reduce the adverse impacts of liquefaction and the relevant construction work are described below.

① Increase of relative density of sandy ground

Sand compaction pile method or vibroflotation method, etc.

② Increase of effective stress

Counterweight embankment method or groundwater lowering method, etc.

③ Quick dissipation of excess pore water pressure

Vertical drain method or sand mat method, etc.

④ Change to liquefaction-resistant material

Replacement method or soil-solidification method, etc.

⑤ Increase of confining pressure of the ground to contain shear deformation

Soil-solidification method, sheet piling method and basal reinforcement method, etc.

3.7 Countermeasures for Rock Fall

3.7.1 Selection of Countermeasures for Rock Fall

The important basic principles for countermeasures for rock fall are the minimisation of damage due to rock fall by installing rock fall prevention works and rock fall protection works, taking the importance of the road, expected scale of rock fall, probability of rock fall occurrence, frequency and actual situation of damage and other relevant matters into consideration and also ensuring of the safety of road transport using the available means, including passage control.

There are two types of countermeasures for rock fall, i.e. (i) rock fall prevention works addressing the sources of rock fall and (ii) rock fall protection works to mitigate the damage by rock fall which has taken place.

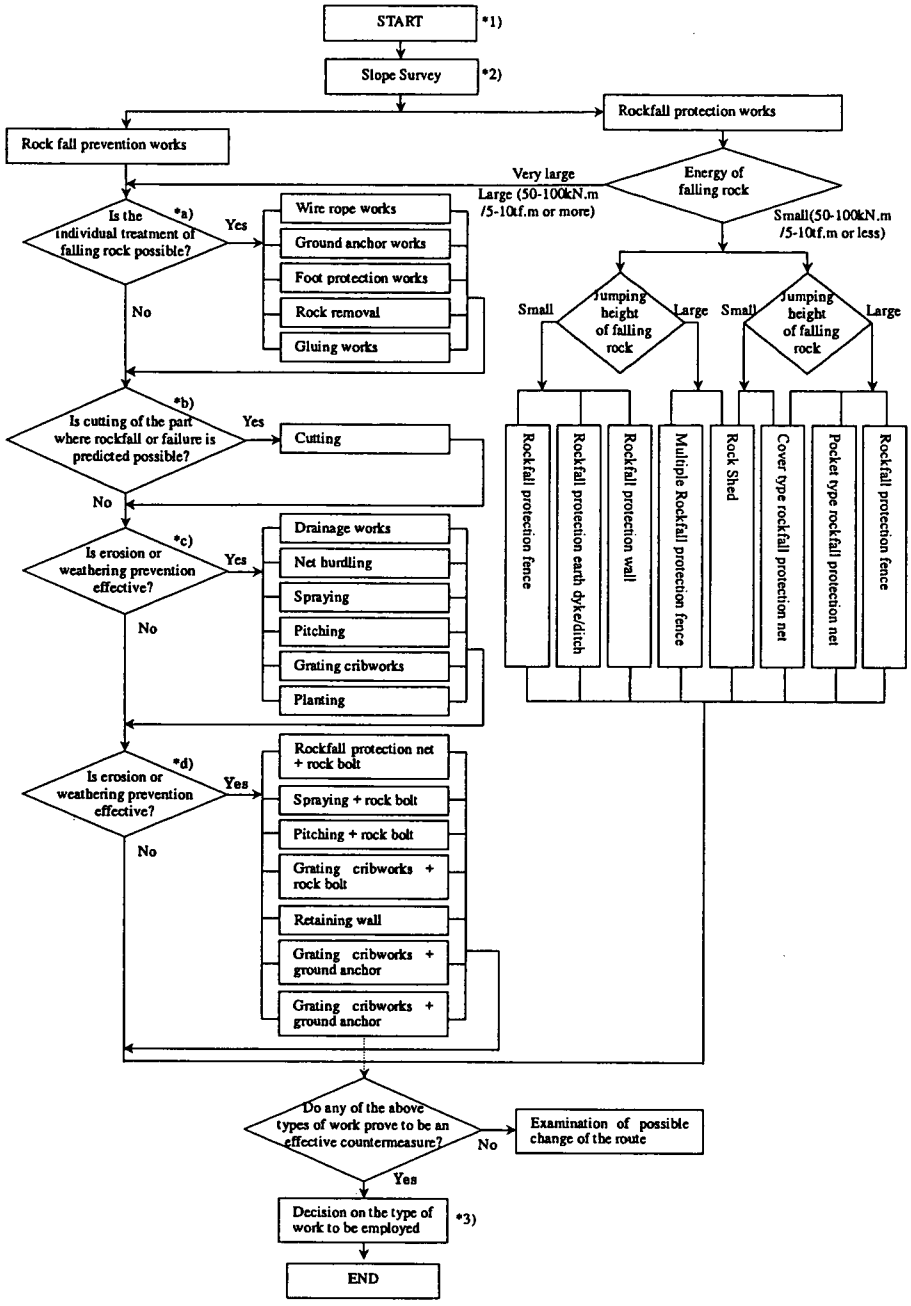
The most basic requirements for the selection of a countermeasure for rock fall are accurate prediction of what type and scale of rock fall is likely to occur at which part of the slope in question and what form of movement the falling rocks are likely to follow and a decision on how to stop these falling rocks or how to let them pass the road harmlessly.

The selection of the most appropriate type of work or combination of different types of work for the road as well as slope conditions at the site is essential by carefully analysing the expected effects, durability, workability, construction cost and maintenance, etc. while referring to Table 3-7-1.

Fig. 3-7-1 is a flow chart which should prove useful for the selection of a suitable countermeasure(s) for rock fall. As this flow chart mainly focuses on the function of each type of work, it is needless to say that the durability, workability, construction cost, maintenance and other relevant conditions must be considered at the same time for the actual selection of a suitable countermeasure(s).

Table 3-7-1 Reference Table for Application of Countermeasures for Rockfall

Category	Type of Work (Characteristics / Legend)	Effects of Countermeasure: Prevention of Weathering /Erosion	Prevention of Occurrence	Change of Direction	Absorption of Energy	Resistance to Inpact	Durability	Maintenance Requirements	Difficulty of Work	Reliability	Cost
	◎	Excellent					Excellent	Low	Easy	Excellent	Inexpensive
	○	Good					Good	Medium	Fair	Good	Varies
	△	Good in some cases					Damage by rockfall	High	Difficult	Good in some cases	Expensive
Rockfall Prevention Work	Cutting		◎				◎	○	△	◎	○
	Removal Works		◎				○	○	△	○	○
	Foot Protection		◎				◎	○	○	◎	○
	Gluing Works	○	○				△	○	◎	△	△
	Ground Anchor		◎				○	◎	○	◎	○
	Wire Rope		◎				○	○	△	○	◎
	Drainage Works	◎					○	○	○	○	◎
	Net Hurdling	○	○	△			○	○	◎	△	◎
	Planting	○	○				○	◎	◎	△	◎
	Shotcrete	◎	○				○	○	◎	○	◎
	Pitching	◎	◎				◎	◎	○	○	◎
	Cribworks	◎	◎				◎	◎	◎	◎	○
	Retaining Wall	◎	◎	△			◎	◎	○	◎	○
	Rockfall Protection Work + Rock Bolt		◎				○	○	◎	○	◎
	Shotcrete + Rock Bolt	◎	◎				○	○	○	◎	◎
	Pitching + Rock Bolt	◎	◎				◎	◎	○	◎	○
	Cribworks + Rock Bolt	◎	◎				◎	◎	○	◎	◎
	Cribworks + Ground Anchor	◎	◎				◎	◎	○	◎	○
	Retaining Wall + Ground Anchor	◎	◎				◎	◎	○	◎	△
Rockfall Protection Work	Cover Type Rockfall Protection Net		○	○	◎		○	○	◎	○	◎
	Pocket Type Rockfall Protection Net			○	○	○	○	○	◎	○	◎
	Rockfall Protecion Fence			◎	○	△	○	○	◎	○	◎
	Multiple Step Rockfall Protection Fence		△	◎	◎		○	○	◎	○	◎
	Rockfall Protection Shelf			◎	◎	◎	◎	◎	○	○	○
	Rockfall Protection Retaining Wall			◎	○	△	◎	○	◎	○	◎
	Rock Shed			◎	◎	◎	◎	◎	○	◎	○
	Rockfall Protection Earth Dyke/Ditch			◎	○	△	◎	○	◎	○	○



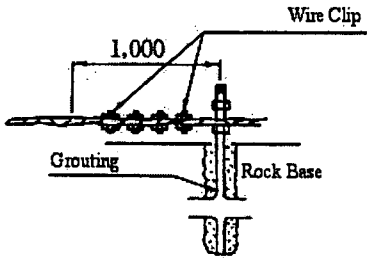
*1) The applicable types of works are simultaneously selected based on the flow and the actual type of works to be employed is then selected from a number of alternatives.
*2) Rockfall prevention works and rockfall protection works must always be compared with each other to decide the suitable type of work.
*3) Table 3-?-? is used to decide the suitable type of works. A combination of defferent types of rock fall prevention worksor rockfall protection works or a combination of rockfall prevention works and rockfall protection works should also be considered.
*a) Suitable for slopes where rockfall and failure independently occur.
*b) Suitable for gentle slopes where transportation of the removed rocks and soil is easy.
*c) Suitable for slopes where relatively small-scale rockfall is expected to occur over a wide area.
*d) Applicable with a combination of rockfall prevention works and rockfall protection works to slopes where relatively large-scale rockfall or failure is expected to occur over a wide area.

Fig. 3-7-1 Selection Flow of Countermeasures for Rockfall

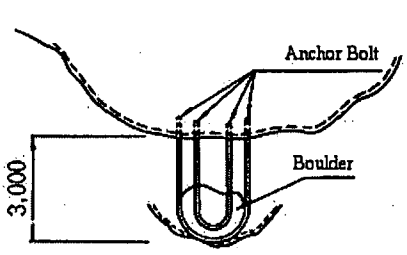
3.7.2 Important Points for Application of Countermeasures for Rock Fall

(1) Rock Fall Prevention Work

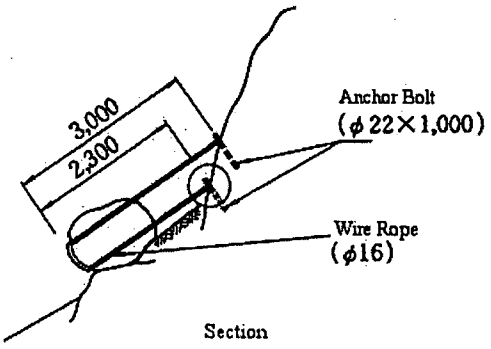
Although rock fall prevention work is a highly effective countermeasure for rock fall, the complete prevention of rock fall using only rock fall prevention work may be difficult. In reality, this type of work should be considered a countermeasure to reduce the frequency of rock fall. Accordingly, it is preferable to employ this type of work in conjunction with a type of rock fall protection work in many cases. Many rock fall prevention works are identical to slope protection works. Examples of rock fall prevention works are shown in Fig. 3-7-2 and Fig. 3-7-3.



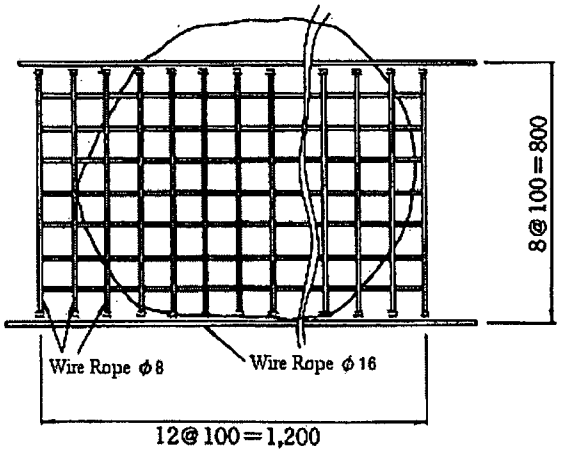
Detailed Drawing of Anchor



Plan



Section



Front Elevation

Fig. 3-7-2 Example of Wire Rope Works

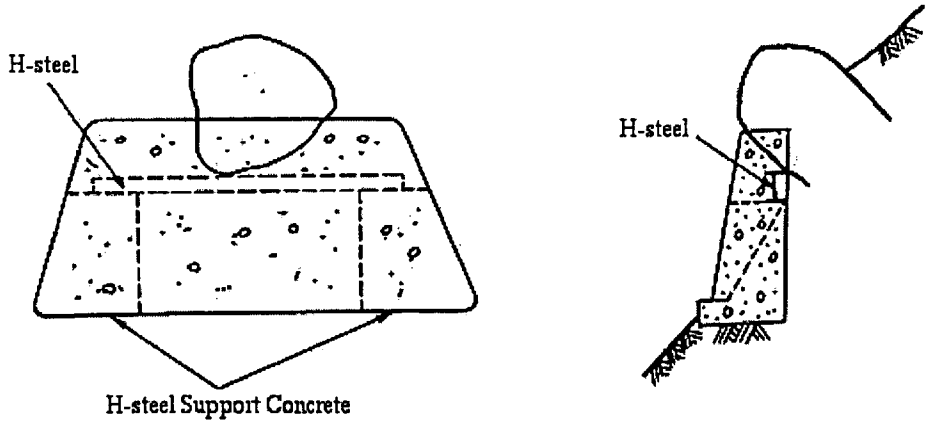


Fig. 3-7-3 Example of Concrete Foot Protection

(2) Rock Fall Protection Work

For the design of rock fall protection work, it is firstly necessary to assume the external forces to be dealt with by a structure. In this context, it must be noted that the weight and velocity of falling rocks and the working direction to and location at a rock fall protection works of falling rocks considerably vary depending on such factors as the topography, geology, degree of weathering of the slope, vegetation and the presence of any other rock fall prevention or protection works at the site. Accordingly, the most suitable values representing these factors must be estimated based on field investigation results and past experience of rock fall.

Rock fall protection works can be designed by either the energy calculation method or the static strength calculation method. In the case of a rock fall protection net and a rock fall protection fence, design calculation at the time of their construction is not necessarily required. Instead, their specifications can be determined based on the specifications used for similar slopes.

1) Rock Fall Protection Net

A rock fall protection net uses such light weight materials as a wire net and wire ropes to entirely cover a slope which has a rock fall hazard and there are two types of such net as described below.

① Cover type rock fall protection net

② Pocket type rock fall protection net

The former intends to confine rocks which have lost their binding power with the ground with friction between the net and the ground and the tensile force of the wire net, producing a function which is similar to

that of rock fall prevention work. In contrast, the latter consists of hanging ropes, posts, wire net and wire ropes. The upper part has an entrance for falling rocks and it has the function of absorbing the energy of falling rocks through collision between the net and the falling rocks.

The design of a pocket type rock fall protection net follows the following processes (Fig. 3-7-4).

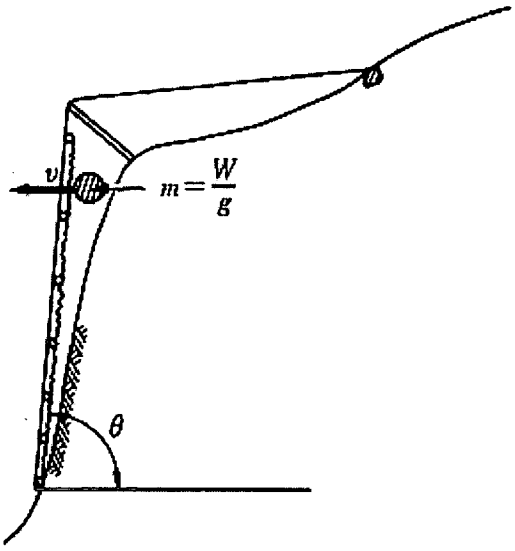


Fig. 3-7-4 Prevention of Impact of Falling Rocks

a) Calculation of energy of falling rock E_w

$$E_w = \frac{1}{2} \cdot \frac{W}{g} (V \sin\theta)^2 \quad \text{..... (3.7.1)}$$

Where,

E_w : energy of falling rock (kN ·m (tf ·m))

W : weight of falling rock (kN (tf))

V : velocity of falling rock (m/sec)

g : gravity acceleration (m/sec^2)

θ : inclination angle of the net (°)

b) Calculation of absorbable energy by pocket type rock fall protection net

$$E_T = E_N + E_R + E_p + E_{HR} + E_L \quad \text{.....(3.7.2)}$$

Where,

E_T : energy absorbed by protection net

E_N : energy absorbed by wire net

E_R : energy absorbed by wire ropes

E_p : energy absorbed by posts

E_{HR} : energy absorbed by hanging ropes

E_L : difference in energy before and after collision

c) If the absorbable energy is found to be larger than the energy of the falling rocks, the stability of the anchor should be examined to withstand the breaking load for the wire ropes.

2) Rock Fall Protection Fence

A rock fall protection fence is an effective countermeasure for relatively small-scale rock fall and the type and dimensions should be determined based on the slope conditions.

Two types of rock fall protection fences exist, i.e. wire rope and wire net type and H-beam type (see Fig. 3-7-5). As the former uses wire ropes and wire net with excellent extensibility, the energy absorption capacity is relatively large. When the installation of a long rock fall protection fence is necessary, it is necessary to divide the entire length into suitable sections for the installation of the fence at each section. In this case, the end parts of the rock fall protection fence in each section should overlap to prevent falling rocks from passing through the gaps between neighbouring fences.

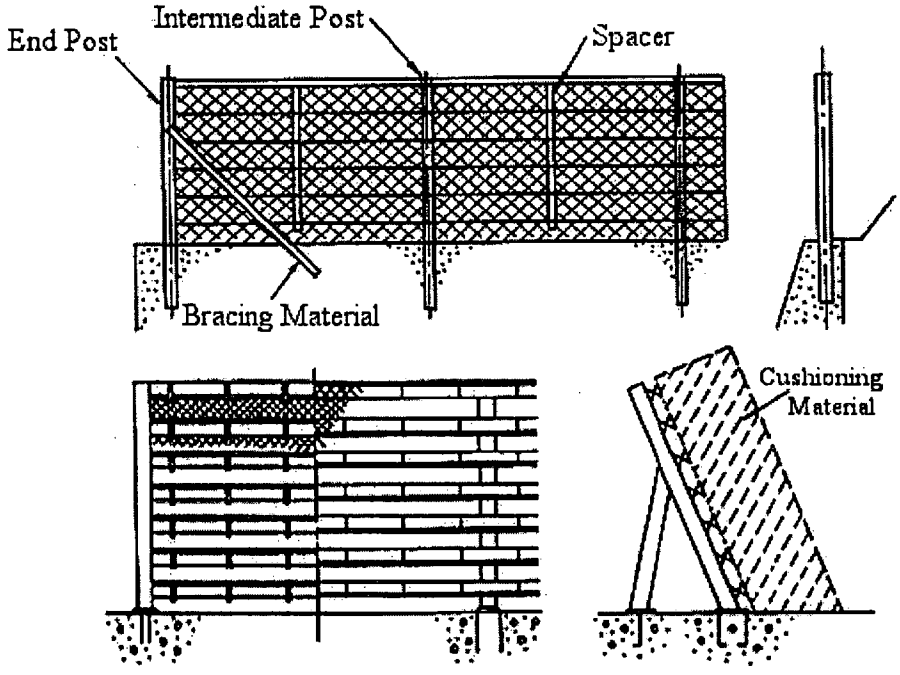


Fig. 3-7-5 Types of Rockfall Protection Fences

Any design of a rock fall protection fence must examine the cross-section and positioning of the members and the stability of its foundations so that the fence can absorb the energy of falling rocks within its allowable displacement.

Only the load of falling rocks is considered for the design of a rock fall protection fence. The collision position with the falling rocks is assumed to be the central point between the posts and two-thirds of the height of the fence from the bottom as shown in Fig. 3-7-6. The remaining one-third of the height is considered to be the height margin. The collision direction of the falling rocks is set at a right angle to the fence.

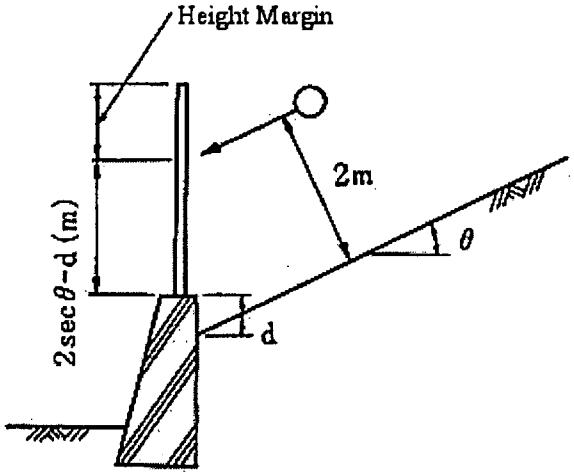


Fig. 3-7-6 Height of Rockfall Protection Fences

The energy of falling rocks (E_i) used for design purposes is calculated by the following formula.

$$E_i = (1+\beta)(1-\mu/\tan\theta)W \cdot H \quad \text{..... (3.7.3)}$$

Where,

E_i : energy of falling rocks (kN·m (tf·m))

β : coefficient of rotating energy (dimensionless)

μ : equivalent friction coefficient of slope (dimensionless)

(value ranging from 0.05 to 0.35 is used depending on the characteristics of the falling rocks and slope)

θ : slope gradient (°)

W : weight of falling rocks (kN (tf))

H : height of fall (m)

The allowable maximum displacement angle of the posts is 15°.

The absorbable energy by a rock fall protection fence (E_T) is calculated by the following formula.

$$E_T = E_R + E_P + E_N \quad \text{..... (3.7.4)}$$

Where,

E_R : energy absorbed by wire ropes

E_P : energy absorbed by posts

E_N : energy absorbed by wire net

3) Rock Shed

A rock shed is used where there is a series of steep slopes with a high rock fall hazard with little room at the roadside, where an expected large-scale rock fall cannot be prevented by a rock fall protection fence or where the high falling height suggests that falling rocks may well jump over a rock fall protection fence.

The design is usually conducted using the allowable stress method by converting the impact load of falling rocks to the static load.

Given the primary objective of rock fall protection work, including a rock shed, of absorbing the kinetic energy of falling rocks, however, it is essential to select a structural form, materials and structural specifications to increase the energy absorbability.

Sand is laid above the roof of the rock shed to act as a cushioning material (sand cushion) with a minimum thickness of 90 cm.

Depending on the site of a rock shed, soil deposition may occur on the roof of the rock shed. In this case, the load of the sedimented soil must be taken into consideration.

Use of the following Formula (3.7.5) is recommended to calculate the impact load of falling rocks when sand is used as a cushioning material.

$$P = 2.108 \cdot W^{2/3} \cdot \lambda^{2/5} \cdot H^{3/5} \cdot \alpha \quad (P = 2.455 \cdot W^{2/3} \cdot \lambda^{2/5} \cdot H^{3/5} \cdot \alpha) \quad \text{..... (3.7.5)}$$

Where,

P : impact force of falling rock (kN (tf))

W : weight of falling rocks (kN (tf))

λ : Lame's constant (kN/m^2 (tf/m^2))

H : height of fall (m)

α : rate of increase based on the ratio between the sand layer thickness and diameter of falling rocks

$\alpha = 1.0(T/D)^{-0.5}$ (however, when $T/D > 1$, $\alpha = 1.0$)

T : thickness of sand layer (m)

D : diameter of falling rocks (m)

It is necessary to carefully determine Lame's constant for use. Recent experiment results suggest that the impact force of weight is similar to the calculated value based on Lame's constant at 1,000 kN/m^2 (100 tf/m) when the laid sand layer thickness is similar to the diameter of the falling rocks. However, Lame's

constant can exceed 1,000 kN/m^2 (100 tf/m) depending on the particle size distribution, water content and degree of compaction of the sand used.

While the head H can be used as the height of the fall of free falling rocks, the height of the fall of falling rocks on a slope with a β gradient can be converted by Formula (3.7.6) (see Fig. 3-7-7).

$$H' = (1 - \mu/\tan\theta)H \quad \text{.....(3.7.6)}$$

Where,

H' : converted height of fall (m)

H : head of slope (m)

θ : gradient of slope (°)

μ : equivalent friction coefficient of slope

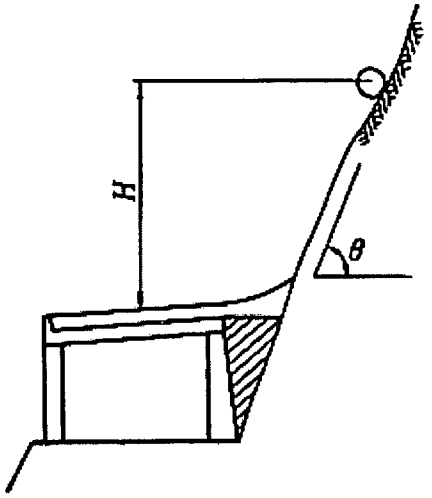


Fig. 3-7-7 Falling Height of Rockfall

4) Rock Fall Protection Retaining Wall

A rock fall protection retaining wall is used as a protection works to prevent rocks from falling onto a road and is often used at sites where the slope gradient at the back is gentle or sites where there is ample room at the roadside.

A rock fall protection retaining wall should be designed to safely absorb the energy of falling rocks with deformation of the wall itself as well as the bearing stratum after calculating the value of such energy.

In addition, it is desirable to establish a pocket at the back of this retaining wall so that fallen rocks as well as fallen soil can be deposited there to a certain extent.

3.8 Countermeasures for Landslides

The route to be planned for the construction of a road must, in principle, avoid areas with a landslide risk. However, if the construction of a road through such an area is necessary because of various planning conditions, the necessary surveys must be conducted to ensure the introduction of sufficient countermeasures for landslides.

3.8.1 Judgement on Degree of Landslide Risk

The degrees of risk of landslide can be classified as in Table 3-8-1 and 3-8-2 and the countermeasures for each classification will be examined by taking account of the following items

Table 3-8-1 Table for Determining the Risk of Landslides

Class of stability	Abnormal states and topographic characteristics	Degree of fluctuation	Precaution for earthworks
A	Block in which main cracks (primary cracks) or toe cracks have occurred, or currently active landslide block with a high potential of giving direct damage to roads, houses or the like regardless of the presence of artificial environmental changes such as cut-embankment and improper drainage. Any portion with new main cracks and side cracks is to be separately classified as special danger A.	Fluctuation a Fluctuation b	As a rule, the route should be modified but, if impossible, countermeasures capable of securing the designed factor of safety should be examined.
B	Block showing no sign of landslide motion, which however shows apparent topographic features of landslide (landslide of colluvial deposit, or weathered rocks) such as the distribution of topographically old slide scarp, has the prime factors of occurrence of landslide, and has a high potential of sliding-out triggered directly by the artificial environmental change. This type of landslide may possibly affect the road facilities or house. Block with a potential of progress but not accompanying toe upheaval. Degree of danger A' for the landslide block with residents and houses in nearby area. Degree of danger B' for the block with a possible occurrence of bedrock landslide.	Fluctuation c	Fill at the head of landslide area and cut at the toe of landslide area is to be avoided if possible. If this is not avoidable, the decrease in factor of safety should be maintained by less than 5% as a rule.
C	Block showing the topographic features of landslide but presently being considered to be stable (no problem unless topographic change is given), so that even if a landslide occurs due to a large-scale environmental change, the landslide is unlikely controlled at the occurrence of the landslide.	There is a potential of occurrence of fluctuation c.	Selection of route is possible at both the head and toe of landslide area but the decrease in the factor of safety must be maintained by less than 10% as a rule.

1) Modification of route should be considered as a rule for the route in the areas where the landslides occur frequently even if a single large scale landslide, of which degree of risk is ranked as A, is expected to occur.

Table 3-8-2 Degree of Landslide Fluctuation

Degree of Landslide Fluctuation		a	b	c	Remarks
Slope Fluctuation	Daily Average (sec.)	5 or higher	1 – 5	Less than 1	3 or higher
	Accumulated (sec/month)	100 or higher	30 – 100	Less than 30	None (Continual Change)
Ground Elongation/Contraction Measured by Invar Wire Extensometer	Daily Displacement (mm)	1 or higher	0.1 – 1	0.02 – 0.1	0.1 or higher
	Accumulated Displacement	10 or higher	2 – 10	0.5 – 2	None (Continual Change)

2) Slight change in the alignment and also countermeasures against landslide are required in landslide areas where the degree of risk is ranked as B.
3) A considerable decrease in the safety factor should be avoided when the earthworks are executed in landslide areas where the degree of risk is ranked as C.

Possible countermeasures for landslides are analysed based on the above principles. The purposes of this analysis should be estimation of the shape of a possible landslide, checking of the stability, feasibility of a minor shift of the route, review of horizontal and longitudinal alignment and the rough examination of countermeasures so that an economical, safe and easy to maintain route and suitable countermeasures can be determined.

3.8.2 Stability Analysis for Landslides

The slicing method is frequently used for stability analysis for landslides. Using this method, the strength along the sliding surface can be determined by either the back calculation method or the soil test. In general, the back calculation method described below is used. According to this method, using a current safety factor of 0.95 – 1.0 for an active landslide and 1.05 – 1.15 for a dormant landslide, a linear expression for the average value of c (cohesion) and $\tan\phi$ over the entire sliding surface is established by the slicing method to produce the relationship chart shown in Fig. 3-8-1.

After deciding the value of c based on Table 3-8-3, it is substituted in the above-mentioned linear expression to determine the value of ϕ. The value of c can be determined by assuming the value of ϕ. In this case, Table 3-8-4 constitutes a useful reference. This table shows the values of c and ϕ as determined by the back calculation method based on actual examples of landslides involving weathered rock which have been caused by road earthwork throughout Japan. The value of ϕ tends to increase in line with a steeper slope gradient.

The target safety factor to ensure road preservation by means of increasing the stability of a slope with a landslide prevention works is called the design safety factor and this factor is determined taking the scale of damage caused by a landslide and the construction cost as well as safety of the works in question into consideration.

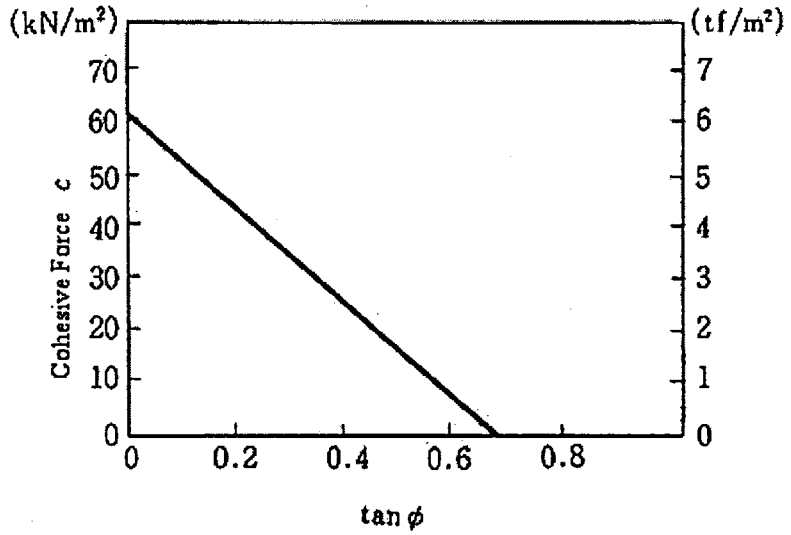


Fig. 3-8-1 Example c-tan ϕ Relationship Chart using Back Calculation Method

Table 3-8-3 Empirical Value of c

Vertical thickness of layer landslide (m)	c (tf/m2)
5	0.5
10	1.0
15	1.5
20	2.0
25	2.5

Table 3-8-4 Range of Sliding Surface Strength of Weathered Rock

Type of Weathered Rock		Cohesion c (tf/m²)	Shearing Resistance Angle ϕ (°)
Metamorphic Rock		0 – 0.2 (0.1)	20 – 28 (26)
Igneous Rock		0 (0)	23 – 36 (29)
Sedimentary Rock	Paleozoic	0 – 0.4 (0)	23 – 32 (29)
	Mesozoic	0 – 1.0 (0.5)	21 – 26 (24)
	Paleogene	0 – 2.0 (0.7)	20 – 25 (23)
	Neogene	0 – 2.5 (2.0)	12 – 22 (12.5)

3.8.3 Countermeasures for Landslides

Landslide countermeasures are largely classified into control works and prevention works. Control works are designed to stop or mitigate landslide activity by changing the natural conditions, including the topography and the state of groundwater. Meanwhile, prevention works are designed to stop part of or all landslide activity using the suppression force of artificial structures.

In many cases, more than one type of landslide countermeasure are used in combination. Table 3-8-5 shows the classification of landslide countermeasures. The landslide characteristics and category should be taken into consideration to enhance the effects of landslide countermeasures. Table 3-8-6 lists the types of landslides and countermeasures. Section 2.7 and Table 2-7-1 should also be referred to for the different types of landslides. Typical landslide countermeasures are described below.

Table 3-8-5 Classification of Countermeasures for Landslide

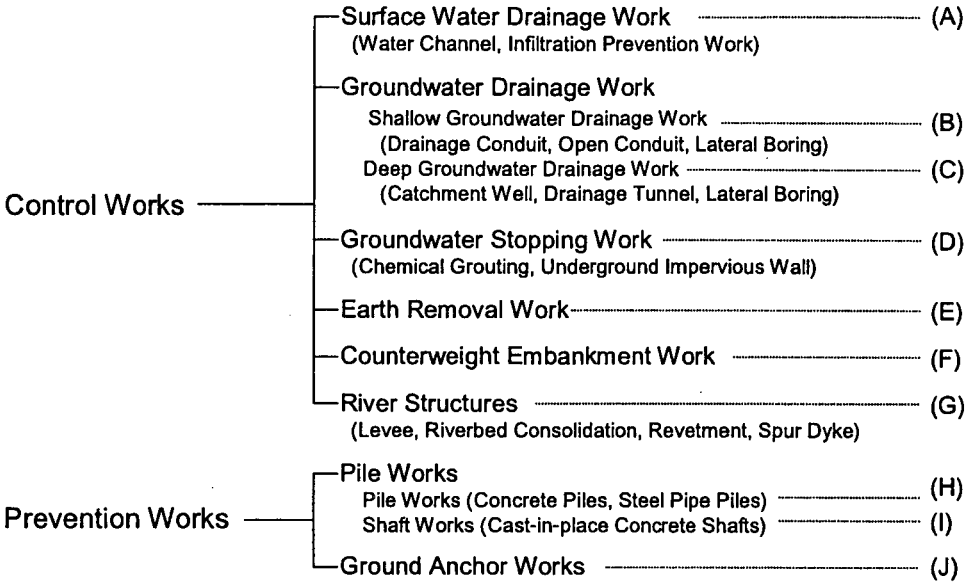


Table 3-8-6 Types of Landslides and Countermeasures

Main Causes		Type of Landslide	Countermeasure										Remarks
			(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)	
Natural Causes	Infiltration of rainfall, increase in groundwater, erosion by river stream	Landslide in bedrock	○	△	◎	△	◎	○	○	◎	◎	○	◎: frequently used ○: often used △: infrequently used
		Landslide in weathered rock	◎	△	◎	△	◎	○	○	◎	◎	○	
		Landslide in colluvial deposit	◎	○	◎	△	○	◎	◎	○	○	△	
		Landslide in clayey soil	◎	◎	○	○	△	◎	◎	△	△	△	
Artificial Causes	Cutting Work	Landslide in bedrock	△	△	○	△	◎	◎	○	◎	○	○	
		Landslide in weathered rock	△	△	○	△	◎	◎	○	◎	○	○	
		Landslide in colluvial deposit	○	○	○	△	◎	◎	○	◎	○	△	
		Landslide in clayey soil	◎	◎	○	△	△	◎	○	○	△	△	
	Filling Work	Landslide in colluvial deposit	△	△	○	△	△	◎	○	◎	◎	○	
		Landslide in clayey soil	△	△	△	△	△	◎	○	○	○	△	

(1) Surface Water Drainage Works

The surface water drainage work is able to drain the seepage water from lakes or ponds and the rain water infiltrated into the landslide area, and comprises the water channel works and infiltration prevention works.

This channel is mainly constructed across a slope to collect rain water and surface water, and its width is relatively large and the water depth may be shallow. This channel is used to quickly and fully drain the collected water to the outside of an area. U-shaped gutter, centrifugal reinforced concrete or corrugated pipes may be used for the draining channel. Bed consolidation work should be performed at every 20 to 30 m to prevent the channel from slipping. For this purpose, the channel may be fixed with wooden stakes if the ground is soft.

(2) Groundwater Drainage Works

This type of drainage works is conducted to reduce the water content and pore water pressure (groundwater level) in the landslide mass by draining the groundwater which flows into or which already exists in the landslide area.

There are two types of groundwater, i.e. shallow groundwater which flows through an aquifer in a colluvial deposit layer and deep groundwater which constitutes confined groundwater in the basement layer. Suitable drainage works for each type of groundwater should be selected by examining the effects of groundwater on a landslide.

1) Drainage Conduit and Open Drainage Conduit

These are the most suited to the drainage of shallow groundwater distributing from the ground surface to the portion about 3m below the ground surface, and are particularly used for draining the groundwater within soil particles in soil layer having a small coefficient of permeability.

Drainage pipe packed with gravel are normally installed every 20 to 30m and are connected to surface water channel or draining conduit. Draining conduit is made of centrifugal reinforced concrete pipes or corrugated pipes and is connected to a drainage channel at the point of bed consolidation.

Drainage conduit is usually installed in valleys or concaves on the ground surface and thus the conduit network often coincides with the surface water draining channel network. Figs. 3-8-2 show the several combinations of open conduits.

2) Lateral Boring Drainage Works

The diameter of the bore hole is 66 mm and, after completing the drilling work, hard vinyl chloride pipe or gas pipe attached with a strainer should be inserted as hole-protecting and water-collecting/draining pipe to an aquifer.

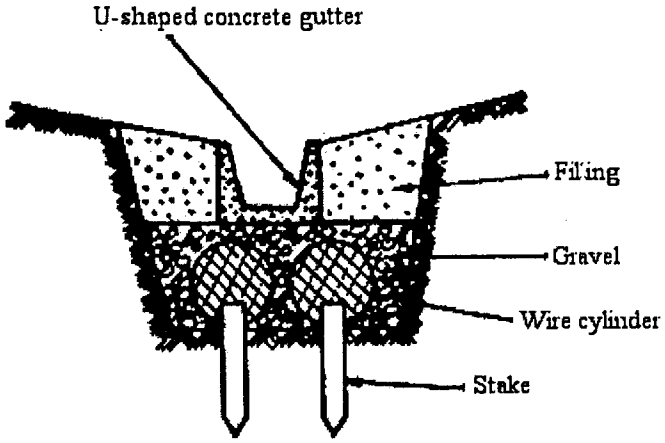
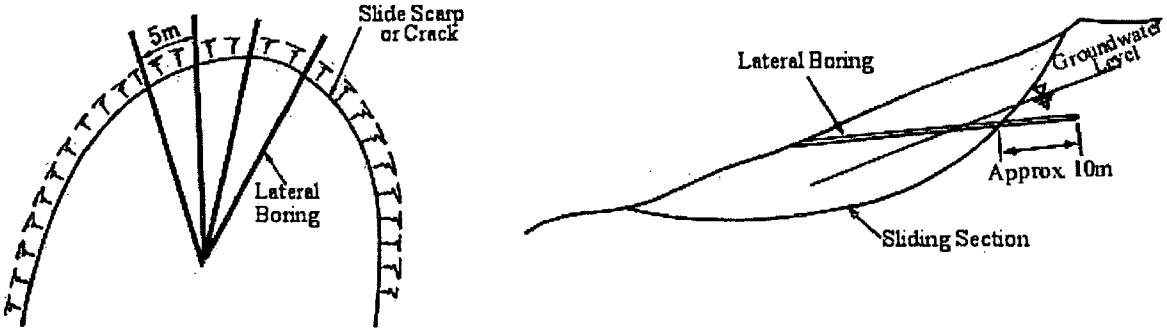


Fig. 3-8-2 Open Conduit



(a) Plan (b)Longitudinal Section

Fig. 3-8-3 Removal of Groundwater by Lateral Boring

The end portion of the drain pipe should be protected by gabion or concrete wall (refer to Fig. 3-8-3).

3) Drainage Well Works

Drainage well is introduced when the required length of lateral boring is too long and when concentrated water drainage is required near the basement layer. It is introduced at a relatively good ground site and has a diameter of mainly between 2.5 m and 4 m. The natural drainage of the collected water through drainage boring works (up to some 100 m in length) or a drainage tunnel is desirable.

The lateral boring works to collect water should be 20 – 50 m in length and 8 – 16 boring holes should be introduced across the sliding surface in a radial manner from the drainage well.

4) Drainage Tunnel

A drainage tunnel is used when the large scale of a landslide and the presence of much groundwater near the surface of the basement layer make it difficult to achieve sufficient drainage by either lateral boring or a drainage well. It is essential for a drainage tunnel to be set up at a stable basement layer below the sliding

surface.

(3) Groundwater Stopping Works

The impervious wall can be built by chemical grouting into ground or by concrete wall constructed by open-cutting to an aquifer, underground water veins are easily gathered at the center, and drainage should be performed by lateral boring.

(4) Earth Removal Works

Normally earth is removed from the head portion of landslide area and not from the tail portion except special cases (see Fig. 3-8-4).

Earth removal works are one method by which the most reliable effects can be expected and are therefore frequently used for medium and small-scale landslides.

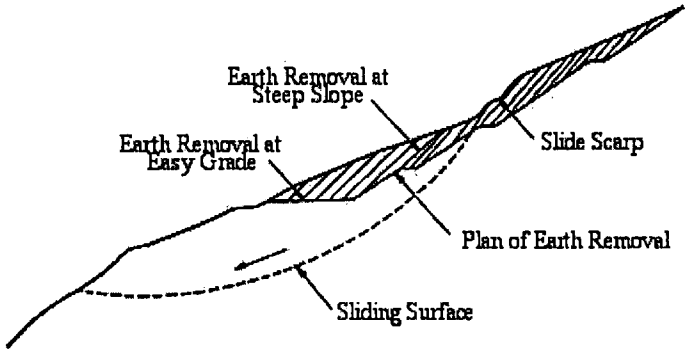


Fig. 3-8-4 Earth Removal Work

(5) Counterweight Embankment Works

Earth is filled on the lower portion of landslide area to stabilize the slope according to the counter weight. However, earth at the tail portion of landslide area is specially disturbed and soft. Thus, if earth is filled near the tail portion, the foundation ground may sometimes collapse or the pore water pressure in landslide mass may increase due to the disturbance of the groundwater flow, thereby making the landslide area unstable. Because of this, the groundwater should be completely drained and then filling works should be performed.

(6) River Structures

Lowering of river beds due to scouring or river bank erosion by running water will frequently degrade the

stability of a slope to result in failures. In this case, the erosion of river bed and bank should be prevented by bed consolidation work, bank protection or spur dike and replacement of watercourse should be considered if necessary.

(7) Pile works

It is desirable to carry out the pile works at a place where the bedrock is strong and the pile is fully able to withstand a moving soil mass. However, if the motion of a landslide is vigorous and the movement exceeds 1 mm per day, the effect of piles cannot be expected since each pile reacts individually unless they have all been placed at the same time in the landslide area. Pile works are not suitable for these areas.

H-section steel piles, reinforced concrete piles and steel pipepiles are used for the insertion method. A bore hole with a large diameter (about 35 to 40 cm) is first driven, and a steel pipe with a diameter of about 30 cm is then inserted to the hole. The inner space of the pipe is then filled up with concrete, and the gap between the pipe and bore hole wall of bore hole is filled with grout.

(8) Shaft Works

Shaft works are used when pile works are not feasible because of the ground conditions or when pile works cannot guarantee the design safety factor because of a high landslide earth pressure. Vertical holes of 2.5 – 6.5 m in diameter are dug in the ground for the placement of reinforced concrete.

(9) Ground Anchor Works

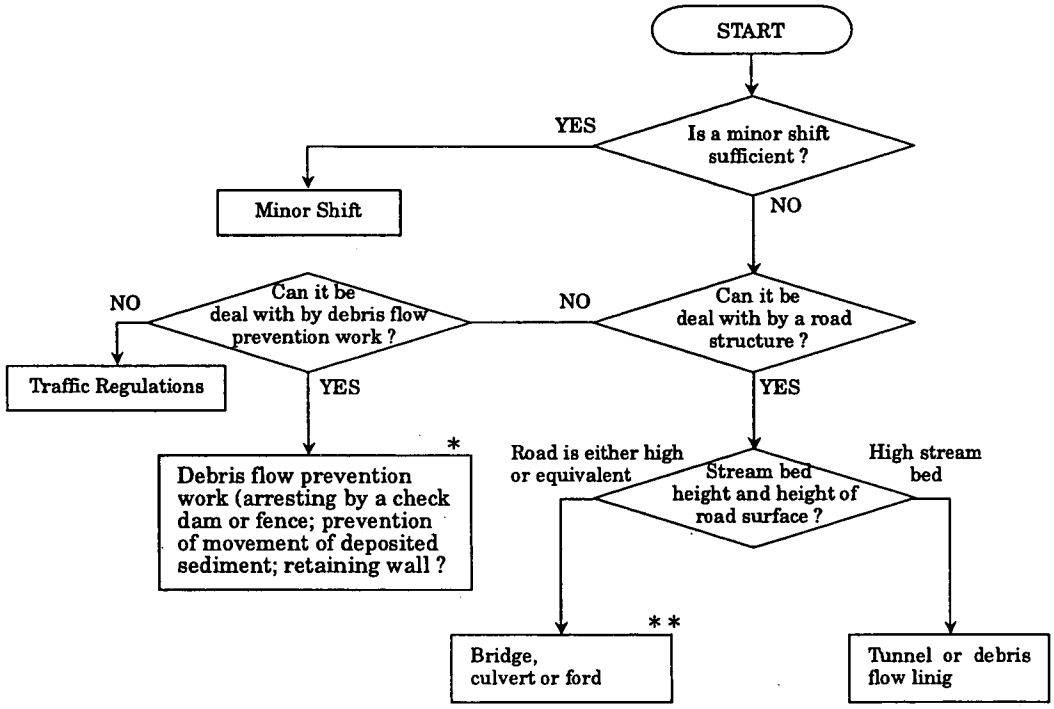
Ground anchor works use high strength steel as the tension member in a relatively small borehole and have a structure whereby bearing plates receive the reaction force. Ground anchor works have the advantage over other types of control works in that they are capable of working a sufficient suppression force by having an initial tensile force prior to major displacement by landslides. They are generally used for small to medium-scale landslides in rock or weathered rock and are often combined with pile or shaft works. They are also used to convey suppression force in association with a retaining wall, cribworks or independent bearing plate, etc.

Careful examination is required in regard to determination of the design load of ground anchor works, way of adding initial tensile force and long-term stability of the fixed part. Moreover, double anti-corrosion treatment must be applied to ensure the durability of ground anchor works.

3.9 Countermeasures for Debris Flow

3.9.1 Selection of Countermeasures for Debris Flow

When a planned road crosses a mountain stream where debris flow is predicted, it is desirable for its route to avoid an area inundated by debris flow. If the crossing of such an area is unavoidable, a suitable countermeasure(s) should be selected and implemented as shown in Fig. 3-9-1 depending on the expected type and frequency of debris flow and the position and elevation of the road surface.



* : Combination with a bridge or culvert should be examined.
** : Combination with traffic regulations should be examined.

Fig. 3-9-1 Flow Chart for Selection of Countermeasure for Debris Flow

For the selection of a countermeasure for debris flow, the expected type (mud flow type debris flow or gravel type debris flow) and frequency of debris flow are firstly considered. In general, when a road crosses an originating or passing area of debris flow along a mountain stream susceptible to debris flow, crossing should, in principle, be provided by a culvert with a sufficient cross-sectional area or a bridge with sufficient clearance. In the case of a mountain stream where high speed mud flow type debris flow is expected to occur at the foot of a volcano or a mountain stream with a high debris flow frequency, a road crossing in the originating or passing area of debris flow should preferably be provided by a bridge with sufficient debris flow clearance, a debris flow shed or a tunnel.

At a site where the gradient of the upstream section of the debris flow sedimentation area is 3 – 10°, the occurrence of debris flow causes considerable fluctuation of the streambed. It is, therefore, desirable not to plan the route passing through such a site and the route should be shifted to either the upstream or downstream and a crossing provided by a bridge with sufficient clearance (see Fig. 3-9-2).

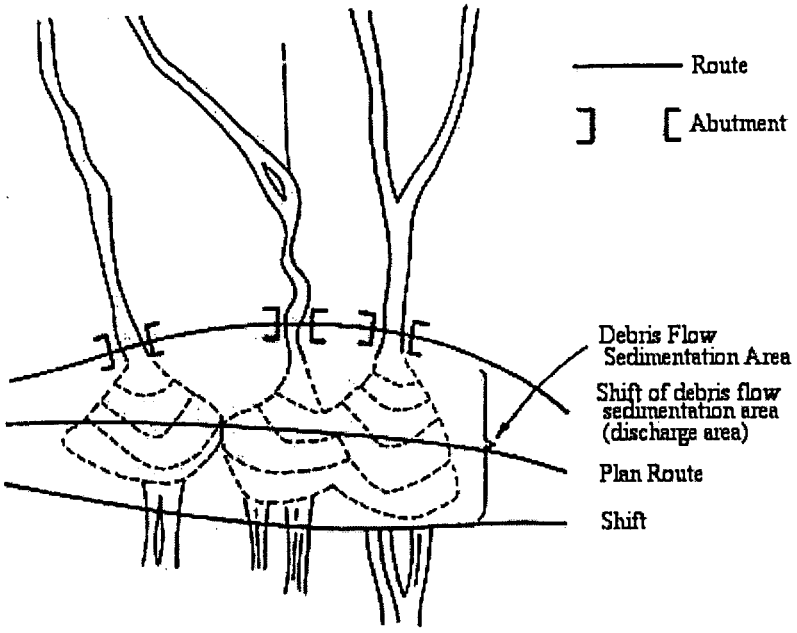


Fig. 3-9-2 Minor Shift in Debris Flow Sedimentation Area

When the road surface is not much higher than the streambed, the introduction of a ford (low level crossing) should be considered. When the road surface is lower than the streambed, the introduction of a debris flow shed should be considered.

If a change of the route or countermeasure is not deemed to be sufficient, it may be necessary to consider the construction of a dam to control debris flow. In the case of this option, it is necessary to fully coordinate with any sabo (erosion control) projects in the area.

For road sections which include an originating and/or passing area of debris flow, passage control should be introduced according to need if the level of rainfall which is high enough to cause debris flow is predicted.

3.9.2 Important Points for Countermeasures for Debris Flow

(1) Estimation of Scale of Debris Flow

For the design of debris flow countermeasures, the volume of sediment, peak discharge, velocity of flow,

water level (wave height), unit volume weight and maximum particle size of the anticipated debris flow should be estimated, if necessary, based on survey results for each mountain stream.

1) Peak Discharge

When a debris flow occurs due to destabilization of sediment deposited on the streambed, the peak debris flow discharge can be estimated using the following formula.

$$Q_{sp} = C_* / (C_* - C_d) \cdot Q_p \quad \text{..... (3.9.1)}$$

where.

Q_{sp} : peak discharge of debris flow (m^3/sec)

Q_p : peak discharge of water alone (m^3/sec)

C_* : volume sediment density of deposited sediment at streambed ($=1-n$; n : void ratio)

C_d : density of debris flow on the move

Meanwhile, the equilibrium sediment density of debris flow is given by the following formula.

$$C_d = \frac{\gamma_w \tan\theta}{(\gamma_s - \gamma_w)(\tan\phi - \tan\theta)} \quad \text{..... (3.9.2)}$$

Where,

γ_s : unit volume weight of sediment (kN/m^3 (tf/m^3))

γ_w : unit volume weight of water (kN/m^3 (tf/m^3)

ϕ : shear resistance angle of deposited sediment at streambed (°)

θ : streambed gradient (°)

The peak discharge of water only Q_p is calculated by the following formula for the critical rainfall causing a debris flow.

$$Q_p = 1/3 \cdot f \cdot r_e \cdot A \quad \text{..... (3.9.3)}$$

Where,

f : peak flow index

r_e : mean rainfall intensity in flood concentration time (mm/hr)

A : catchment area (km^2)

2) Velocity and Highest Water Level (Wave Height)

A Manning type uniform flow formula is used for the velocity of flow based on debris flow observation results in Japan.

$$v = \frac{1}{n} \cdot h^{2/3} (\sin\theta)^{1/2} \quad \text{..... (3.9.4)}$$

Where,

v : mean velocity of debris flow (m/sec)

h : wave height of debris flow (m)

The coefficient of roughness n to be used is approximately 0.03 for a fixed bed water channel and 0.1 for a movable bed.

The highest high water level can be calculated using the peak discharge, a velocity-wave height formula and the width of the streambed in the passing area of debris flow.

(2) Bridge

The planned cross-section of a bridge should allow passing of the peak debris flow discharge. The height of the bridge girders is determined by adding a clearance to the debris flow wave height. It is preferable not to place the bridge piers on the streambed. Special attention must be paid to avoiding narrowing of the channel width at the bridge site. Even if it is necessary to place piers on the streambed, their positions should avoid the central section of the channel so that the piers are not destroyed by debris flow.

(3) Culvert

The planned cross-section of a culvert should allow passing of the peak debris flow discharge and both the height and horizontal width must exceed double the maximum particle size of the gravel contained in debris flow. The axis of a culvert, including the water channel upstream, should be as straight as possible to coincide with the flow direction of debris flow. Careful attention should be paid to avoiding a smaller cross-sectional area and gradient of the water channel downstream than those at the culvert.

Further attention must also be paid to the possible clogging of the culvert by driftwood. When the outflow of a large quantity of driftwood is anticipated, it is desirable to set up a boom(s) in the upstream (Fig. 3-9-3).

(4) Ford

When there is little head between the streambed and the road surface in an originating or passing area of debris flow, the road should be given a structure (ford) which cannot be destroyed by the passing of debris flow over the road surface (Fig. 3-9-4).

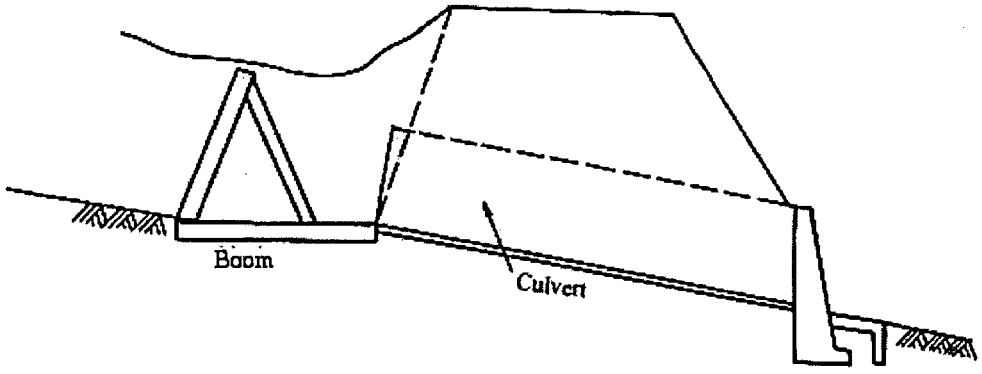


Fig. 3-9-3 Culvert and Boom

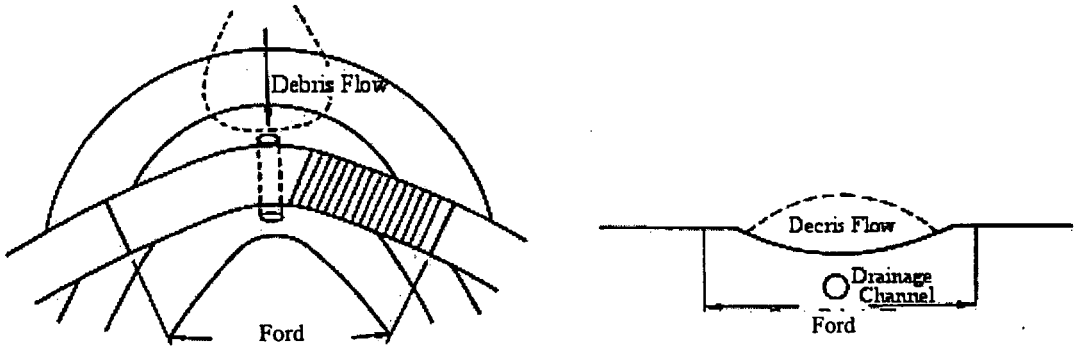


Fig. 3-9-4 Ford

(5) Debris Flow Shed

The structure of a debris flow shed should be similar to that of a rock shed. The longitudinal gradient along the flow direction in the channel should, in principle, be similar to the streambed gradient in the upstream so that there is no soil deposition above the shed. The width of the flow area should be identical to the width of the streambed in the upstream.

The side wall should have a height equivalent to the debris flow wave height plus a margin height and should be designed to guide debris flow from a mountain stream to the shed.

(6) Capture of Discharged Soil by Dam and Fence

One or several dams or fences can be used to catch an entire debris flow or large gravel and driftwood so that only soil and water, which can be drained by road drainage facilities in the downstream, are allowed to flow further downwards. The design sedimentation gradient should be half of the current streambed gradient. Concrete or steel permeable dams and fences are used to reduce the sedimentation volume during ordinary floods. The size of the opening should be less than 1.5 times of the maximum diameter of large gravel to

catch debris flow (Fig. 3-9-5).

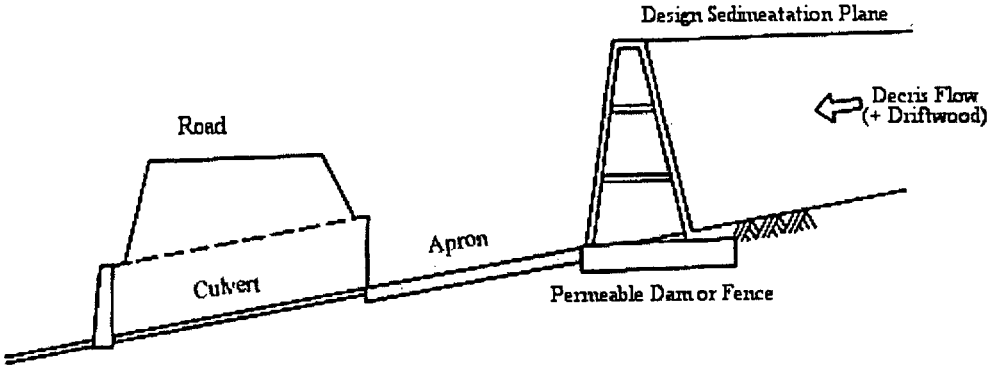


Fig. 3-9-5 Capture of Discharged Soil by Permeable Dam or Fence

3.10 Retaining Wall

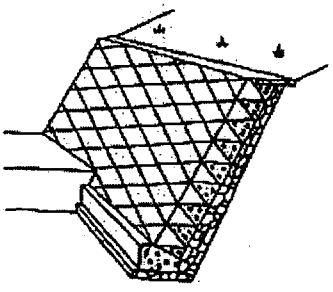
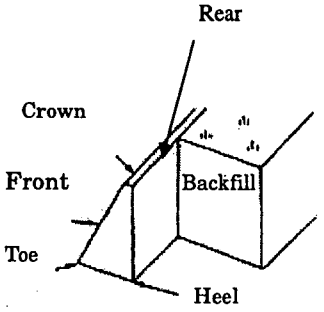
A retaining wall is a structure which is constructed to stabilize the soil or to protect a slope surface when banking or cutting cannot be conducted in accordance with the standard cross section because of constraints posed by the right-of-way and/or topography. Accordingly, it is essential to design an appropriate structure for a retaining wall based on a comprehensive judgement on the topographical, geological and work conditions.

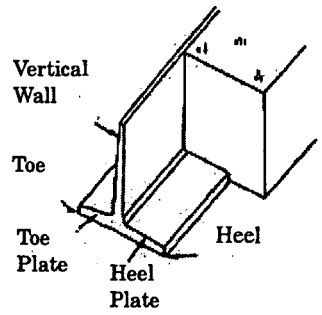
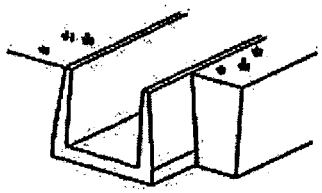
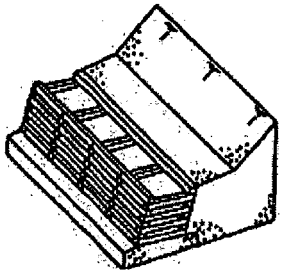
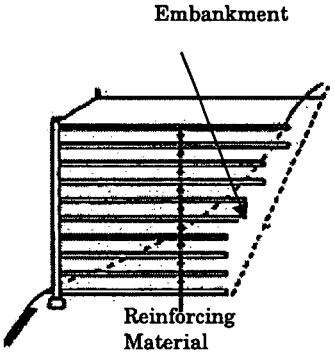
3.10.1 Selection of Structural and Foundation Types

There are many types of structures for retaining walls and Table 3-10-1 shows the criteria relating to the application height, characteristics and important points for application, etc. for the selection of a suitable structural type.

The types of foundations for a retaining wall are principally classified into spread foundations and pile foundations. The preferable type of foundations for a retaining wall are spread foundations in view of their movement together with the bearing stratum and the embankment at the back. Even if the surface layer is soft, spread foundations tend to be used with the replacement or improvement of the soft layer provided that the supporting layer exists at a relatively shallow depth from the ground surface (approximately 2 – 3 m). Pile foundations are used when the application of spread foundations is difficult.

Table 3-10-1 Types of Retaining Walls and Criteria of Selection

Type	Shape	General Application Height	Characteristics	Important Points for Application
Concrete Block (Stone) Masonry Retaining Wall		▪ Up to 7 m (the gradient and backfill thickness change depending on the height) ▪ Possibly up to 15 m when large blocks are used	▪ Used for the prevention of small-scale failure at the lower part of a slope or for the protection of slope surface	▪ Used when the earth pressure is low due to compacted ground at the back or good back soil ▪ Relatively inferior structure in terms of the aseismatic performance
Gravity Retaining Wall		▪ Up to approx. 5 m	▪ In principle, the cross-section should ensure that the horizontal load is supported by its own height, not generating tensile stress to the structure's cross-section	▪ Used at a site with good supporting ground in view of the large reaction force of the base plate ▪ Unsuitable for pile foundations

Cantilever (Inverted T-Shape/L-Shape/ Inverted L-Shape) Retaining Wall		▪ Approx. 3 – 10 m	▪ The vertical wall resists the horizontal load as a cantilever ▪ The weight of the earth above the heel plate can be used to stabilise the retaining wall	▪ Applicable in the case of pile foundations ▪ Many precast products are available
U-Shape Retaining Wall			▪ Integral side walls and base plate; used for canal roads, etc. ▪ Strats may be used between the side walls	▪ Often used below groundwater level, making it necessary to examine the impacts of the water pressure and stability against buoyancy ▪
Crib Retaining Wall		▪ Up to approx. 15 m	▪ Cribworks of precast concrete members or similar with hollow parts filled with a filling material; offers good permeability ▪ Resists horizontal load with the weight of the members and filling material	▪ Designed in a similar manner to a leaning type retaining wall
Reinforced Soil Retaining Wall		▪ Approx. 3 – 18 m	▪ The walls are formed by reinforcing earth with friction between the reinforcing material and earth or supporting pressure by an anchor plate; there are many construction methods	▪ Some deformation occurs to ensure the reinforcement effect ▪ While spread foundations can be employed at relatively soft ground, sufficient analysis of the stability of the entire structure, etc. is necessary
Other Types of Retaining Walls	The appropriate type of retaining wall is adopted depending on many constraining conditions, including the topography, geology, soil properties, working conditions and surrounding environment.			

3.10.2 Determination of Design Conditions

(1) Determination of Soil Parameter

1) Parameters Used for Calculation of Earth Pressure, etc.

① Parameters for shear strength of soil

The parameter for shear strength of soil is obtained from either the unconfined compression test or the triaxial compression test. The empirically inferred value using Formula (3.10.1) or Formula (3.10.2) based on the value of N obtained by the standard penetration test can also be used.

- Cohesion c of clayey soil

$$c = 6N \sim 10N \ \ (\mathrm{kN/m^2}) \quad (0.6N \sim 1.0N \ \ (\mathrm{tf/m^2})) \qquad (3.10.1)$$

- Shear resistance angle ϕ of sandy soil

$$\phi = 15 + \sqrt{15 \cdot N} \leq 45°, \ \ N > 5 \qquad (3.10.2)$$

If it is difficult to conduct the soil test for retaining walls of up to 8 m in height, the empirical value shown in Table 3-10-2 can be used.

Table 3-10-2 Shear Strength Parameters of Backfilling Soil

Type of Backfilling Soil	Shear Resistance Angle (ϕ)	Cohesion (c) [2]
Gravelly Soil [1]	35°	-
Sandy Soil	30°	-
Clayey Soil (W_L <50%)	25°	-

Note: 1) In case of pure sand, it is possible to use the value of gravelly soil
2) In case that soil parameter is assumed with this table, the value of cohesion should be neglected.

② Unit volume weight of soil

The unit volume weight of soil γ ($\mathrm{kN/m^3}$ ($\mathrm{tf/m^3}$)) used for calculation of the earth pressure is established using a sample of the soil used for back-filling (banking). If it is difficult to conduct the soil test for retaining walls of up to 8 m in height, the value shown in Table 3-10-3 can be used instead of conducting the soil test.

Table 3-10-3 Unit Volume Weight of Soil (Unit: kN/m3 (tf/m3))

Type of Ground	Soil Type	Loose Soil	Dense Soil
Natural Ground	Sand and Gravel	18 (1.8)	20 (2.0)
	Sandy Soil	17 (1.7)	19 (1.9)
	Cohesive Soil	14 (1.4)	18 (1.8)
Embankment	Sand and Gravel	20 (2.0)	
	Sandy Soil	19 (1.9)	
	Cohesive Soil (w_L <50%)	18 (1.8)	

Note: The value achieved by subtracting 9 kN/m³ (0.8 tf/m³) from the value in the table can be used as the unit volume weight of soil below the groundwater level.

2) Parameters Used for Calculation of Bearing Capacity of Foundations

① Allowable bearing capacity of ground

The allowable bearing capacity of the ground is, in principle, determined by conducting an in-situ test. When it is difficult to conduct an in-situ test for retaining walls of up to 8 m in height, the value shown in Table 3-10-4 can be used.

② Friction angle ϕ_B and adhesion c_B between foundation base and ground

When the shear parameters c and ϕ of the bearing stratum are established by the soil test and investigation, the friction angle of the foundation base ϕ_B is determined to be $\phi_B = \phi$ for cast-in-place concrete retaining walls and $\phi_B = 2/3 \cdot \phi$ for precast concrete retaining walls.

The adhesion c_B between the base plate and the ground should be determined taking disturbance to the ground during construction work into consideration.

When it is difficult to conduct a soil test, etc., the value shown in Table 3-10-5 can be used.

Table 3-10-4 Types of Bearing Stratum and Allowable Bearing Capacity (Normal Value)

Type of Bearing Stratum		Allowable Bearing Capacity q_a (kN/m^2) (tf/m^2))		Remarks		
				q_u (kN/m^2 (kgf/cm^3)		N Value
Bedrock	Uniform Hard Rock with Few Cracks	1,000	(100)	>10,000	(>100)	-
	Hard Rock with Many Cracks	600	(60)	>10,000	(>100)	-
	Soft Rock/Mudstone	300	(30)	>1,000	(>10)	-
Gravel Layer	High Density	600	(60)	-		-
	Low Density	300	(30)	-		-
Sandy Ground	High Density	300	(30)	-		30 – 50
	Medium Density	200	(20)	-		20 – 30
Cohesive Soil Ground	Very Hard	200	(20)	200-400	(2.0-4.0)	15 – 30
	Hard	100	(10)	100-200	(1.0-2.0)	10 - 15

Table 3-10-5 Friction Coefficient and Adhesion Between Foundation Base and Ground

Condition of Shearing Face	Type of Bearing Ground	Friction Coefficient $\mu = \tan \phi_B$	Adhesion C_B
Rock/Gravel and Concrete	Bedrock	0.7	Not considered
	Gravel Layer	0.6	Not considered
Laying of Rubble or Crushed Stone between Foundation Ground and Foundation Concrete	Sandy Soil	0.6	Not considered
	Cohesive Soil	0.5	Not considered

Note: In the case of precast concrete, the friction coefficient is regarded as not exceeding 0.6 even if the foundation is bedrock

(2) Materials Used and Allowable Stress

1) Concrete

In principle, concrete of which the strength is equal to or higher than the design strength shown below should be used.

- Plain concrete : 18 N/mm^2 (180 kgf/cm^2)
- Reinforced concrete : 21 N/mm^2 (210 kgf/cm^2)

The allowable compressive stress, allowable shear stress and allowable bond stress of the concrete used for reinforced concrete members are those shown in Table 3-10-6. However, the allowable bond stress is only applied to reinforcing bars of which the diameter is up to 51 mm.

Table 3-10-6 Allowable Stress of Concrete (Unit: N/mm^2 (kg/cm^2))

Type of Stress	Design Reference Strength of Concrete	21 (210)	24 (240)	27 (270)	30 (300)
Compressive Stress	Bending Compressive Stress	7 (70)	8 (80)	9 (90)	10 (100)
	Axial Compressive Stress	5.5 (55)	6.5 (65)	7.5 (75)	8.5 (85)
Shear Stress	Bearing of Shearing Force by Concrete Alone	0.36 (3.6)	0.39 (3.9)	0.42 (4.2)	0.45 (4.5)
	Bearing of Shearing Force Jointly by Concrete and Diagonal Tensile Reinforcement	1.6 (16)	1.7 (17)	1.8 (18)	1.9 (19)
	Punching Shear Stress	0.85 (8.5)	0.9 (9.0)	0.95 (9.5)	1.0 (10)
Adhesive Stress	Against Deformed Reinforcing Bar	1.4 (14)	1.6 (16)	1.7 (17)	1.8 (18)

2) Reinforcing Bars

The allowable stress for reinforcing bars of up to 51 mm in diameter is as shown in Table 3-10-7.

Table 3-10-7 Degree of Allowable Stress for Reinforcing Bars (Unit: N/mm^2 (kg/cm^2))

Degree of Stress and Type of Member		Type of Reinforcing Bar	SD295A SD295B	SD345
Degree of Tensile Stress	Case where the effect of an impact load or earthquake is not included in a combination of loads	Members under normal circumstances [1)]	180 (1,800)	180 (1,800)
		Members under a severe environment [2)]	160 (1,600)	160 (1,600)
	Basic value of allowable stress when a combination of loads includes the effect of an impact load or earthquake		180 (1,800)	200 (2,000)
	Case where a lap joint length or fixed length of reinforcing bar is to be calculated		180 (1,800)	200 (2,000)
Degree of Compressive Stress			180 (1,800)	200 (2,000)

Notes: 1) Applicable to a normal environment or permanently positioned in either water or soil.

2) Applicable to cases where drying and wetting are repeated more frequently than in a normal environment and where the members are placed in soil below the level of groundwater containing harmful substances.

3) Increase of Allowable Stress by Combination of Loads

The allowable stress to consider the effect of an earthquake, wind load and collision load is calculated by multiplying the value of allowable stress stipulated in Table 3-10-6 and Table 3-10-7 by an increasing coefficient corresponding to the specific combination of loads as shown in Table 3-10-8.

Table 3-10-8 Additional Coefficient for Degree of Allowable Stress

Combination of Loads	Additional Coefficient
When considering the effect of an earthquake	1.50
When considering the effect of wind load	1.25
When considering the effect of collision load	1.50

(3) Load

For the design of a retaining wall, the various types of loads listed below should generally be taken into consideration. However, it is not necessary to consider all types of load depending on the various conditions at the site of a retaining wall and the type of structure.

① Dead load; ② Live load; ③ Earth pressure; ④ Effects of earthquake; ⑤ Water pressure and buoyancy; ⑥ Snow load; ⑦ Wind load; ⑧ Collision load

The dead load used for a cantilever type retaining wall includes the weight of the soil by regarding soil above the heel plate as part of the body in addition to the body weight. The live load is considered to constitute the surcharge for design purposes and its value is generally set at $q = 10$ kN/m^2 (1.0 tf/m^2).

(4) Effects of Earthquakes

1) Basic Concept

The inertia force originating from the self weight of a retaining wall and the earth pressure of the back-filling soil during earthquake must be considered as the effects at the time of an earthquake. However, past experience teaches us that the careful implementation of design and construction work under normal design conditions should ensure sufficient resistance of retaining walls to ordinary earthquakes without specifically considering the effects of earthquakes. Accordingly, analysis of the stability vis-à-vis earthquakes can be omitted for ordinary retaining walls of up to 8 m in height. Analysis of the stability against earthquakes should be conducted for retaining walls of up to 8 m in height when the importance of individual retaining walls and the difficulty of their restoration demand such analysis.

Table 3-10-9 provides reference levels of design earthquake motion to be considered for stability analysis vis-à-vis earthquakes.

Table 3-10-9 Design Earthquake Motion for Analysis of Stability at Time of Earthquake

Degree of Importance	Ease/Difficulty of Restoration	
	Difficult	Easy
Important	To conduct aseismicity analysis (assuming medium-scale earthquake motion; large-scale earthquake motion is assumed if there is a risk of very important secondary damage)	To conduct aseismicity analysis (assuming medium-scale earthquake motion)
Other	To conduct aseismicity analysis (assuming medium-scale earthquake motion)	To conduct aseismicity analysis (assuming medium-scale earthquake motion; this analysis can be omitted for retaining walls of up to 8 m in height)

2) Horizontal Design Seismic Coefficient

The horizontal design seismic coefficient when the effects of earthquakes are considered should be calculated using Formula (3.10.3).

$$k_h = c_z \cdot k_{h0} \quad \text{..... (3.10.3)}$$

Where,

k_h : horizontal design seismic coefficient

k_{h0} : standard horizontal design seismic coefficient: Table 3-10-10 can be used

c_z : regional revision coefficient

Table 3-10-10 Standard Values of Design Lateral Seismic Coefficient

Type of Ground	Type I	Type II	Type III
To deal with medium-scale earthquake motion	0.12	0.15	0.18
To deal with large-scale earthquake motion	0.16	0.20	0.24

The regional revision coefficient to be used depends on the regional category classified based on the earthquake frequency and is 1.0 for Region A, 0.85 for Region B and 0.7 for Region C.

The ground types for aseismatic design are, in principle, based on the characteristic value T_G of the ground as shown in Table 3-10-11. When the ground surface is identical to the basement layer, it is classified as Type I ground.

The characteristic value T_G of the ground is calculated by Formula (3.10.4).

$$T_G = 4\sum(H_i/V_{si}) \quad \text{..... (3.10.4)}$$

Where,

T_G : characteristic value of the ground (sec)

H_i : thickness of the i[th] soil layer (m)

V_{si} : mean shear elastic wave velocity at the i[th] soil layer (m/sec);

this value is determined by Formula (3.10.5)

< Case of clayey soil layer >

$$V_{si} = 100N_i^{1/3} \quad (1 \le N_i \le 25) \quad \text{..... (3.10.5)}$$

< Case of sandy soil layer >

$$V_{si} = 80N_i^{1/3} \quad (1 \le N_i \le 50)$$

N_i : mean value of N of i[th] soil layer as determined by the standard penetration test

i : code number of i[th] soil layer from the ground surface when the ground in question is divided into n layers between the ground surface and the basement layer.

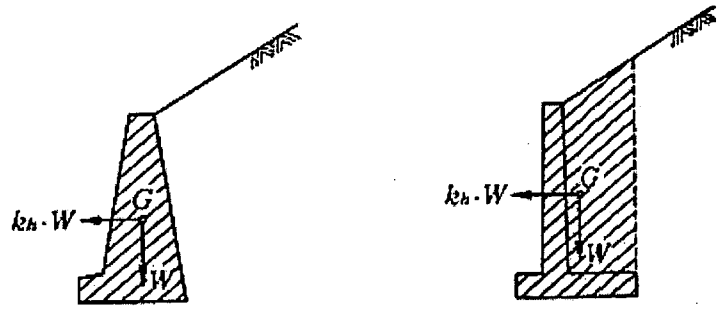
Here, the basement layer means the upper face of the soil layer of which the value of N is not less than 25 for a clayey soil layer and not less than 50 for a sandy soil layer or the upper face of the soil layer of which the shear elastic wave velocity is not less than approximately 300 m/sec.

Table 3-10-11 Ground Types for Aseismatic Design

Type of Ground	Characteristic Coefficient of the Ground T_G (S)
Type I	$T_G < 0.2$
Type II	$0.2 \leqq T_G < 0.6$
Type III	$0.6 \leqq T_G$

2) Calculation of Inertia Force at Time of Earthquake

The inertia force at the time of an earthquake is calculated by multiplying the self weight with W by the horizontal design seismic coefficient k_h and is assumed to work horizontally, passing through the centre of gravity G of the cross-section of the body. In the case of cantilever retaining walls, the area which includes the body and the soil mass above the heel plate is assumed to form the cross-section of the body for determination of the position of the centre of gravity (Fig. 3-10-1).



(a) Gravity Retaining Wall (b) Cantilever Retaining Wall

Fig. 3-10-1 Inertia Force at Time of Earthquake

3.10.3 Earth Pressure

As a retaining wall is a structure which is in contact with the earth, it is subject to earth pressure to the wall. The earth pressure caused by about-to-collapse back soil due to the forward movement of the wall, i.e. in the direction away from the embankment, is called the active earth pressure. As the purpose of a retaining wall is to support an about-to-collapse soil mass, it is generally designed based on this active earth pressure.

The earth pressure used at the design stage of a retaining wall varies depending on whether an embankment or a cut slope forms its back. There is the additional question of whether or not the effects of an earthquake should be taken into consideration. Refer to 3.10.2-(4) for the judgement on whether or not earthquake effects should be considered.

(1) Earth Pressure Acting on Retaining Wall for Embankment

The earth pressure acting on the retaining wall for an embankment is calculated by the trial wedge method. The acting surface of the earth pressure is, in principle, the back face of the concrete body (Fig. 3-10-2). In the case of a cantilever retaining wall, the base substantially extends to the back and the soil above the base is believed to show the same behaviour as a retaining wall when displacement of the retaining wall occurs. For this reason, the line vertically stretching from the heel of the base is assumed to form the virtual back face for the analysis of stability (Fig. 3-10-3).

For the wall friction angle δ, the value in Table 3-10-12 is used while the value between soil and soil in Table 3-10-12 are used for the wall friction angle at the virtual back face of a cantilever retaining wall.

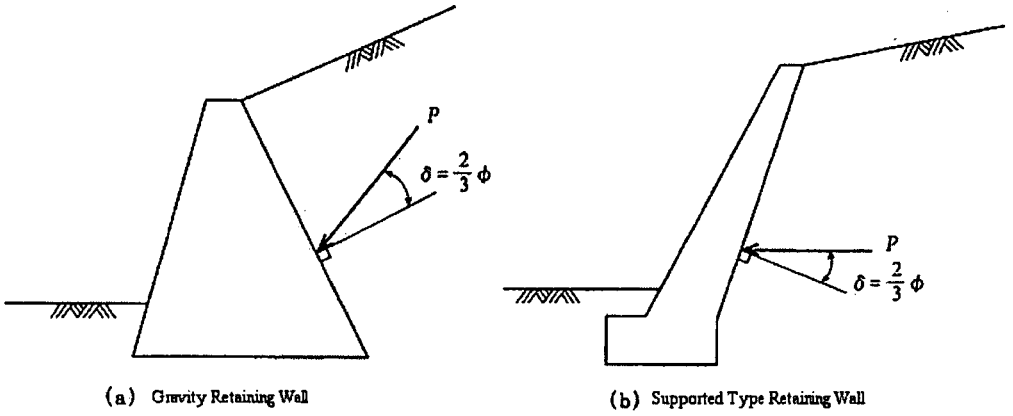
Earth pressure calculation should follow the processes below (see Fig. 3-10-4).

① Assumption of the sliding surface

② Calculation of the earth wedge weight and analysis of the balance of forces

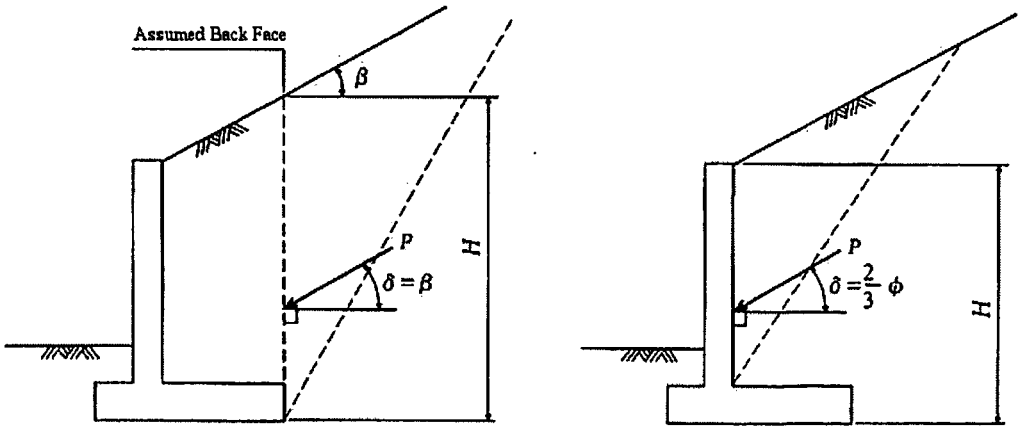
③ Calculation of the maximum value for P (resulting force of the active earth pressure P_A) by changing the subject sliding surface

The acting position of the active earth pressure force P_A is at the centre of gravity of the earth pressure distribution. Normally, the earth pressure distribution can be assumed to be a triangular distribution and, in this case, the acting position is one-third of the distribution height H from the bottom of the earth pressure distribution.



(a) Gravity Retaining Wall (b) Supported Type Retaining Wall

Fig. 3-10-2 Acting Surface of Earth Pressure and Friction Force on Wall Surface in Case of Gravity Retaining Wall , etc.



(a) Acting Surface of Earth Pressure for Stability Analysis (Assumed Vertical Back Face)

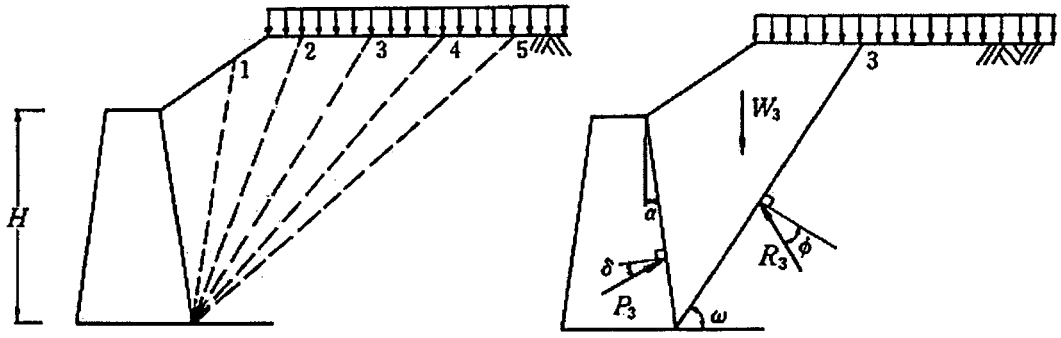
(b) Acting Surface of Earth Pressure at Time of stress Check of Member

Fig. 3-10-3 Acting Surface of Earth Pressure and Assumed Sliding Line in Case of Cantilever Retaining Wall

Table 3-10-12 Wall Friction Angle

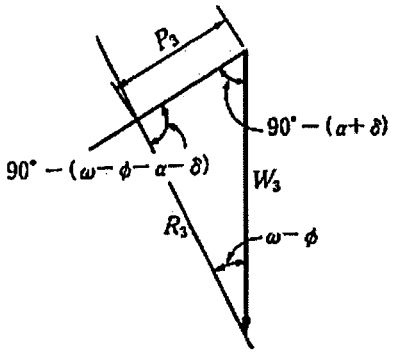
Type of Retaining Wall	Subject of Analysis	Type of Friction	Wall Friction Angle
Gravity Type Leaning Type, atc.	Stability	Soil and Concrete	$\delta = 2\phi / 3$
	Member Stress		
Cantilever Type Counterfort Type, etc.	Stability	Soil and Soil	$\delta = \beta$ (see Fig.3-10-3)
	Member Stress	Soil and Concrete	$\delta = 2\phi / 3$

Note: When $\beta > \phi$, $\delta = \phi$



(a) Trial Wedge

(b) Assumed Earth Wedge (position of sliding line : 3)



(c) Link Polygon

W_3 : Size and derection are known

P_3, R_3 : Only the derection is known

$$P_3 = \frac{W_3 \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

W : Weight of the earth wedge (including live load) (kN/m (tf/m))

R : Reaction force acting on the sliding surface (including live load) (kN/m (tf/m

P : Resulting earth pressure (including live load) (kN/m (tf/m))

α : Angle formed by the back surface of the wal and vertical plane (°)

ϕ : Shear resistance angle of backfilling soil (°)

δ : Friction angle of wall surface (°)

ω : Angle formed by the Assumed sliding surface and horizontal plane (°)

Fig. 3-10-4 Trial Wedge Method

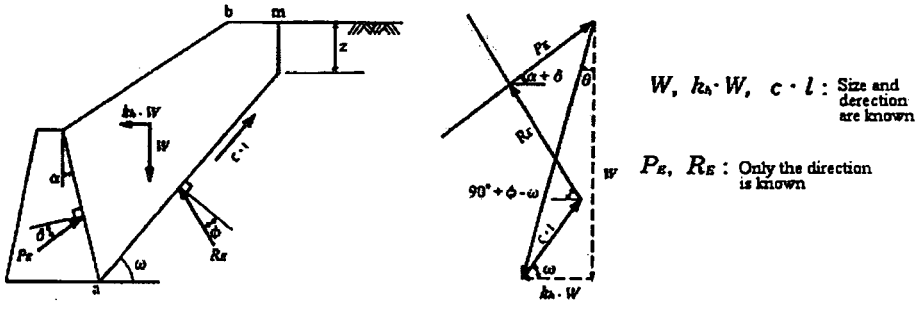
(2) Earth Pressure Acting on Retaining Wall for Cut Slope

As the main purpose of this type of retaining wall is to protect a cut slope, either a leaning type retaining wall or a concrete block retaining wall is mainly used. The earth pressure acting on a leaning type retaining wall differs from that acting on a retaining wall for an embankment as it involves a cut face, i.e. an abnormal boundary surface. To be more precise, the earth pressure in this case is affected by the position and inclination of such boundary surface, roughness of the boundary surface, state of drainage, width of the

back-fill section and properties as well as degree of compaction of the back-filling material. Accordingly, it is essential for the estimated earth pressure acting on a retaining wall for a cut slope to appropriately reflect the site conditions. The trial wedge method is used to calculate the earth pressure as in the case of the earth pressure acting on a retaining wall for an embankment.

(3) Calculation of Earth Pressure at Time of Earthquake

For calculation of the earth pressure at the time of an earthquake using the trial wedge method, the lateral inertia force at the time of an earthquake is assumed to act on the assumed earth wedge as shown in Fig. 3-10-5 and then a link polygon considering such lateral inertia force should be solved. The acting position of the earth pressure is assumed to be the point at one-third of H from the lower face of the base for the retaining wall. The wall friction angle δ is assumed to be half of ϕ when the earth pressure is directly acting on a concrete wall.



(a) Assumed Earth Wedge (b) Link Polygon

k_A : design lateral seismic coefficient

θ : seismic composite angle $\theta = \tan^{-1} k_A$ (°)

c : cohesion (kN/m² (tf/m²))

l : assumed length of sliding line (m)

z : cohesion height (m)

$$z = \frac{2c}{\gamma} \cdot \tan(45° + \frac{\phi}{2})$$

γ : unit volume weight (kN/m³ (tf/m³))

ϕ : shear resistance angle (°)

Fig. 3-10-5 Calculation Method of Earth Pressure at Time of Earthquake

3.10.4 Stability Analysis for Retaining Wall

With respect to the stability of retaining wall, items 1), 2) and 3) should be examined but, depending upon the site conditions and the scale of retaining wall, items 4) ~ should be also examined. (Fig. 3-10-6)

1) Stability against sliding
2) Stability against overturning
3) Stability for bearing capacity of foundation ground
4) Entire Stability including backfill and foundation ground

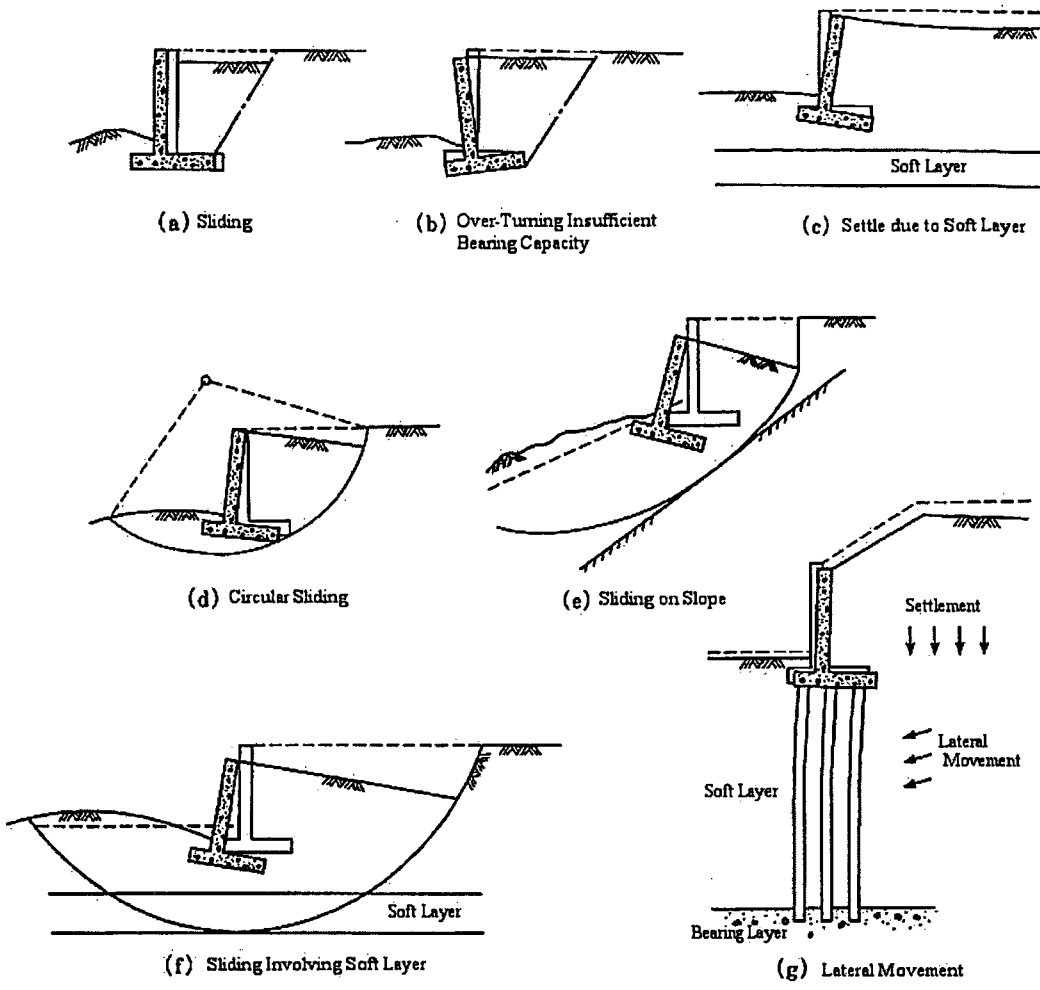


Fig. 3-10-6 Types of Retaining Wall Failure

(1) Stability against Sliding

The force, which tends to slide the retaining wall along the plane below the base, is the horizontal component of the earth pressure and is resisted by the shear resisting force created between the foundation ground and base. The factor of safety against sliding should satisfy the following formula:

$$F_s = \frac{\text{Resisting force against sliding}}{\text{Sliding force}} = \frac{\left(\sum V \cdot \mu + c_B \cdot B\right)}{\sum H} \quad \text{..... (3.10.6)}$$

where

$\sum V$: total vertical load on the bottom of the base

$\sum H$: total horizontal load on the bottom of the base

μ : coefficient of friction between base and foundation ground

c_B : adhesion between base and foundation ground

B : width of base of retaining wall (m)

The safety factor shall not be less than 1.5 at the ordinary condition and 1.2 at time of earthquake.

The width of base should be increased for stabilization if the factor of safety F_s of equation (3.10.6) is not able to satisfy. However, if this is not possible because of restrictions such as topographic conditions, the depth of embedment should be increased in order to consider the passive earth pressure at the front, or projection at the bottom of base should be applied to increase horizontal resistance.

(2) Stability against Overturning

Load due to the self weight of retaining wall, surcharge and earth pressure will act to the bottom of base of retaining wall. The ground reaction below the bottom will vary depending upon the location of point to which the resultant of these loads works. In Fig. 3-10-7, the distance d (m) from the toe to this point can be expressed by

$$d = \frac{\sum Mr - \sum Mo}{\sum V} = \frac{W \times a + Pv \times b - P_H \times h}{W + Pv} \quad \text{..... (3.10.7)}$$

Where,

$\sum M_r$: moment of resistance at toe of base of retaining wall (kN·m/m (tf·m/m))

$\sum M_o$: overturning moment at toe of base of retaining wall (kN·m/m (tf·m/m))

$\sum V$: total vertical load at the bottom of base (kN/m (tf/m)

Distance of eccentricity e from the center of the base of the point of application of resultant R can be expressed by

$$e = \frac{B}{2} - d \quad \text{..... (3.10.8)}$$

As the condition of stability against overturning, the acting position of the resultant must be

- Case at ordinary condition : $|e| \le B/6$ (within central one-third of the width of the base)
- Case at seismic condition : $|e| \le B/3$ (within central two-thirds of the width of the base)

(3) Stability for Bearing Capacity of Foundation Ground

Ground unit reaction can be derived from following equations. (Refer to Fig. 3-10-7)

$$\begin{matrix} q_1 \\ q_2 \end{matrix} = \frac{W + Pv}{B}(1 \pm \frac{6e}{B}) \qquad \text{..... (3.10.9)}$$

The ground reaction at seismic condition should be determined by equations (3.10.10) and (3.10.11).

With $e \le \frac{B}{6}$ $\quad q_1 = \frac{Pv_E + W}{B}(1 + \frac{6e}{B})$ ..... (3.10.10)

With $\frac{B}{6} \le e \le \frac{B}{3}$ $\quad q_1 = \frac{2(Pv_E + W)}{3d}$ ..... (3.10.11)

Also q_1 and q_2 must satisfy the following formula:

$$\begin{matrix} q_1 \\ q_2 \end{matrix} \le q_a = \frac{q_u}{Fs} \qquad \text{..... (3.10.12)}$$

Where

P_v : vertical component of earth pressure resultant at ordinary condition

P_{vE} : vertical component of earth pressure resultant during earthquake

q_a : allowable bearing capacity of ground

q_u : ultimate bearing capacity of ground

F_s : Factor of safety for bearing capacity of ground

($F_s = 3$ for ordinary condition; $F_s = 2$ during an earthquake)

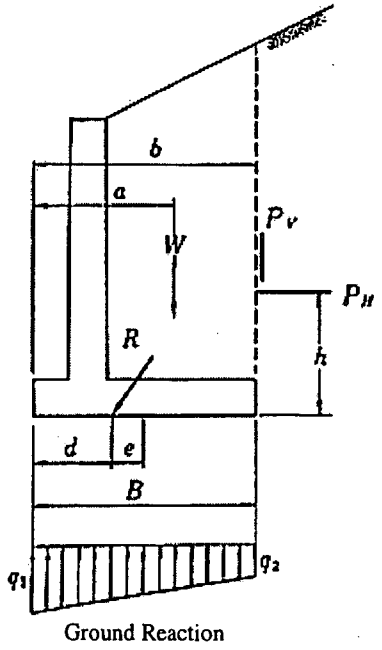


Fig. 3-10-7 Point of Resulting Force and Ground Reaction

(4) Entire Stability, Including Back Embankment and Foundation Ground

In the case of a retaining wall construction on ground incorporating a soft layer, the load of the embankment at the back of the wall may cause failure inside the ground, consolidation settlement by the embankment and/or lateral flow of the ground. The stability against such failure should be confirmed by analysing the possibility of circular sliding and other relevant matters.

When a retaining wall is constructed on a slope, it is necessary to analyse the stability of the entire slope, including the embankment at the back. Multiple retaining walls are occasionally constructed on a slope. In this case, it is essential to confirm the entire stability of the slope in addition to analysis of the stability of individual retaining walls.

3.10.5 Design of Various Types of Concrete Retaining Walls

(1) Concrete Block Retaining Wall, Masonry Retaining Wall

The general configuration of a concrete block retaining wall is shown in Fig. 3-10-8. Concrete blocks have mainly been used in recent years because of the shortage of masonry stone.

While the design of a concrete block (stone) retaining wall (thickness of blocks: approximately 35 – 50 cm) can be conducted in accordance with the slope gradient in Table 3-10-13 which is determined empirically in correspondence with the height, attention should be paid to the application height. Back-filling gravel is used to reduce the water pressure on the block (stone) wall by discharging water at the back and also to prevent a pressure increase at the back of the retaining wall by dispersing the load acting on the blocks. Accordingly, the back-filling gravel should have good permeability, such as crusher run. The standard thickness of the back-filling gravel at the upper part is 30 cm.

The use of multiple-step concrete block (stone) retaining walls should, in principle, be avoided because of the problem concerning the entire stability of the foundation ground, including the retaining walls. To be more precise, such negative developments as acting of the weight of the upper retaining walls as surcharge on the lower retaining walls and the concentration of drained water from the upper walls to a specific part of the lower walls are conceived in the case of this type of retaining wall.

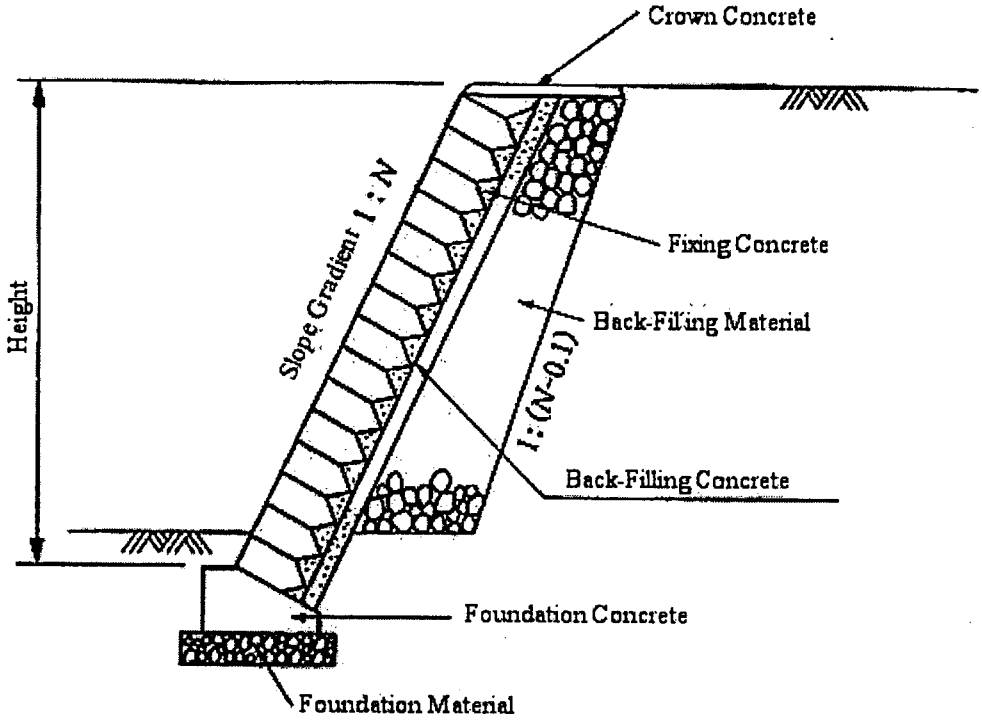


Fig. 3-10-8 Typical Concrete Block Masonry Retaining Wall

Table 3-10-13 Relationship between Wall Height and Gradient of Wall

Height (m)		0 – 1.5	1.5 – 3.0	3.0 – 5.0	5.0 – 7.0
Gradient of Wall	Embankment	1: 0.3	1: 0.4	1: 0.5	1: 0.6
	Cut Slope	1: 0.3	1: 0.3	1: 0.4	1: 0.5
Backfilling Concrete Thickness (cm)		5	10	15	20

(2) Gravity Retaining Wall

This type will support the earth pressure by means of its self weight, and the wall should be designed in such a manner that the resultant of earth pressure and self weight will not create tensile stress in the body of wall because the body usually consists of plain concrete (see Fig. 3-10-9).

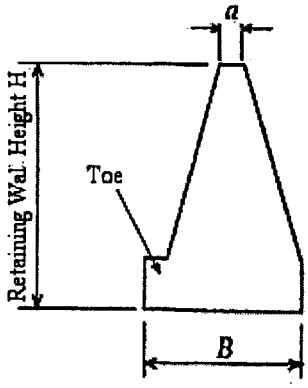


Fig. 3-10-9 Shape of Gravity Retaining Wall

In determining dimensions of the wall, it is desirable for the width B of the base to become about 0.5 to 0.7 times the height of the retaining wall.

(3) Cantilever Retaining Wall

Cantilever retaining wall comprises a vertical wall and a bottom slab and each member resists external forces as cantilever beams. Depending on the position of the vertical wall, it is classified into the inverted-T type, L type and inversed L type. The suitable type is selected based on the site conditions although a well-balanced inverted-T type is normally used (Fig. 3-10-10).

This type of retaining wall is adopted for a wide range of heights, partly because the stability of the wall is ensured by its reinforced concrete structure and back-filled soil above the heel plate which acts as the self weight and partly because it uses less concrete than a gravity retaining wall.

Its members are designed in a cantilever style where the connecting parts of the vertical wall, toe plate and heel plate form the fixed ends.

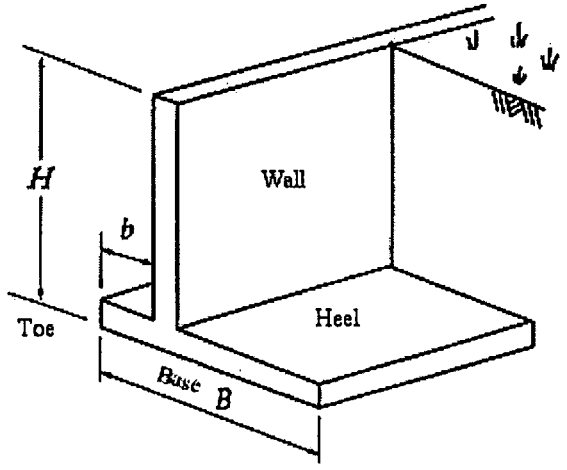


Fig. 3-10-10 Shape of Inverted T Type Retaining Wall

3.10.6 Design of Foundations

As a retaining wall is often constructed in a place with complex topography and geology, the foundations must be carefully designed.

Two types of foundations, i.e. spread foundations and pile foundations are feasible for a retaining wall. Spread foundations are almost always used for sites where the topography changes. One crucial aspect of such spread

foundations is the embedment depth of the retaining wall and the following points must be taken into consideration.

① In the case of a concrete block retaining wall, at least the base stones should be buried in the ground.

② In the case of a concrete retaining wall, the embedment depth should be at least some 50 cm from the upper face of the toe plate.

Even if there is an outcrop of a good quality bearing stratum, a sufficiently embedment depth should be secured in view of possible scouring of the ground in the future.

3.10.7 Design of Drainage

Another important point in the design of a retaining wall is drainage. When the water content of the soil at the back increases with the infiltration of water, the earth pressure rises due to the increased density and reduction of the internal friction angle and cohesion of the soil and, in the case of clayey soil, swelling due to added moisture. Meanwhile, when the groundwater level rises, hydrostatic pressure is added to the earth pressure, constituting a cause of the destabilization of the retaining wall.

Various types of drainage, including weep holes, ditch type drainage and continuous back drainage, can be available for retaining walls. Although there are many combinations for actual application, weep holes should always be introduced. Weep holes are commonly introduced at the rate of one weep hole per 2 – 3 m^2 of retaining wall using a PVC pipe.

3.10.8 Design of Reinforced Soil Wall

(1) General

A reinforced soil wall is a soil retaining structure constructed by means of laying reinforcing materials in an embankment.

The reinforcing mechanism employed by this type of retaining wall is that the earth pressure acting on an almost vertical wall is balanced by pull-out resistance force of the tensile reinforcement materials laid inside an embankment. Many methods with different reinforcing materials and different wall structures have been proposed for practical application (Fig. 3-10-11). Typical reinforcing materials include steel strip, grid or sheet-shaped geotextiles using polymeric materials and steel bar with an anchor plate. The former uses

friction resistance to reinforce soil while the latter relies on bearing resistance to achieve the same effect.

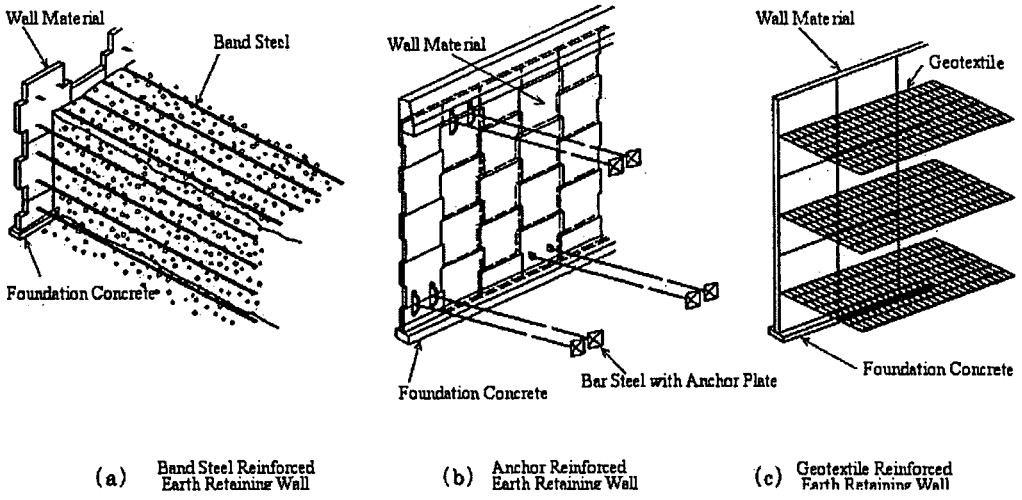


Fig. 3-10-11 Schematic Drawing of Typical Reinforcement Soil Walls

The characteristics of reinforced soil walls include: (i) an embankment with an almost vertical wall can be constructed at a restricted right of way in an urban or mountainous area, (ii) spread foundations can be used at a relatively soft bearing stratum where a conventional retaining wall would require pile foundations provided that sufficient technological analysis is conducted and (iii) they enjoy excellent aseismicity (see Fig. 3-10-12).

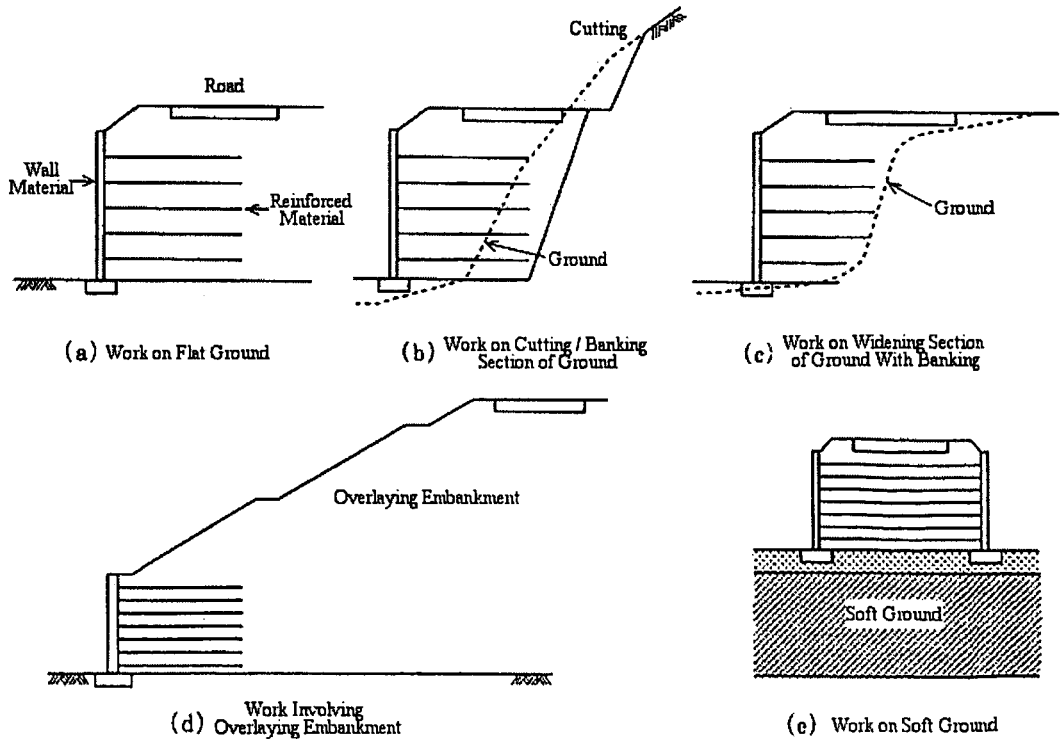


Fig. 3-10-12 Application Examples of Reinforcement Soil Wall

Another characteristic of reinforced soil walls is that some deformation is required to achieve the reinforcing effect. Moreover, long-term durability defined as, for example, physical or chemical stability in the case of a geotextile reinforcing material or corrosion resistance in the case of a steel reinforcing material, must be guaranteed to ensure the function of a retaining wall as a permanent structure.

(2) Design Principles

Any design of a reinforced soil wall must assume the types of failure shown in Fig. 3-10-13, analyse the items described below and secure the required design values as well as safety factor. Stability analysis vis-à-vis earthquakes should also be conducted when deemed necessary.

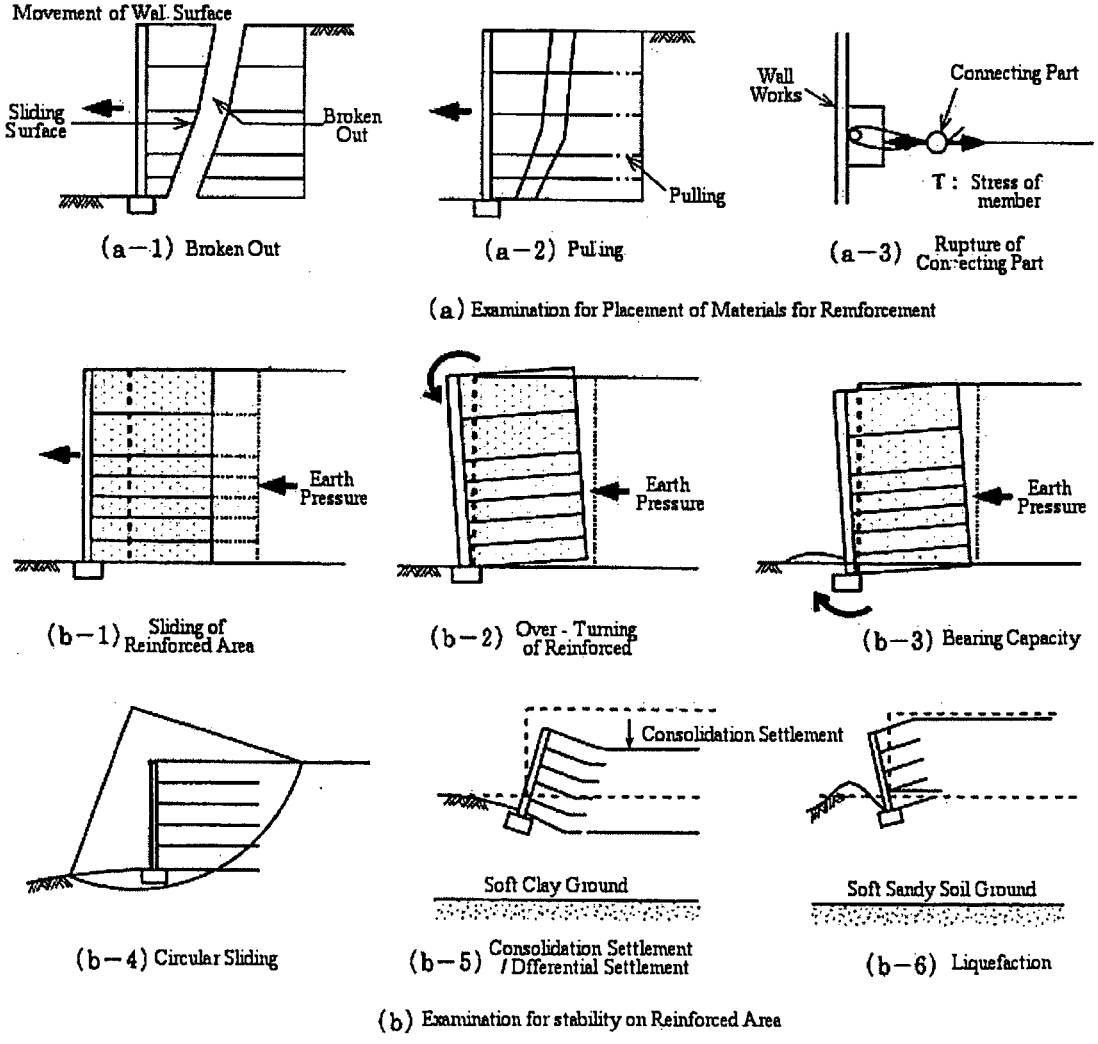


Fig. 3-10-13 Types of Failure and Examination Items for Design of Reinforcement Soil Wall

1) Arrangement of Reinforcing Material

① Broken-Out of Reinforcing Material

The earth pressure acting on the wall of the reinforced soil wall is firstly calculated. Next, based on this lateral earth pressure, the required resistance for the reinforcing material is calculated in correspondence with the placement position and interval of such material (see Fig. 3-10-14). A type of reinforcing material of which the design tensile strength (allowable stress) exceeds the required resistance is then selected. Stress checking for the connecting part between the reinforcing material and the wall is also necessary.

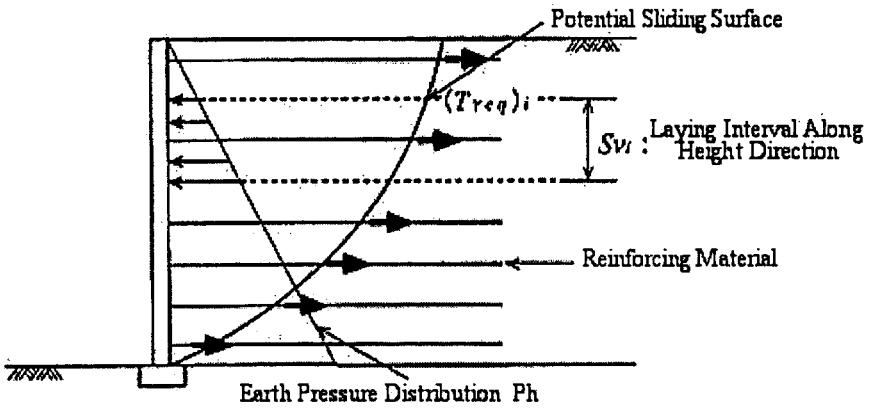


Fig. 3-10-14 Idea of Calculating Required Resistance Force for Reinforcing Material (Example of Geotextile Reinforced Soil Wall)

② Pull-out of Reinforcing Material

The basic length of the reinforcing material should, in principle, be determined to ensure that the pull-out resistance force T_p to be provided by the reinforcing material in a stable soil mass (resistance area) situated at the inner side of the potential sliding surface in an embankment exceeds the value calculated by multiplying the resistance force (T_{req}) required to prevent the reinforcing material from being pulled out by the earth pressure acting on the wall by the safety factor F_s (Fig. 3-10-15).

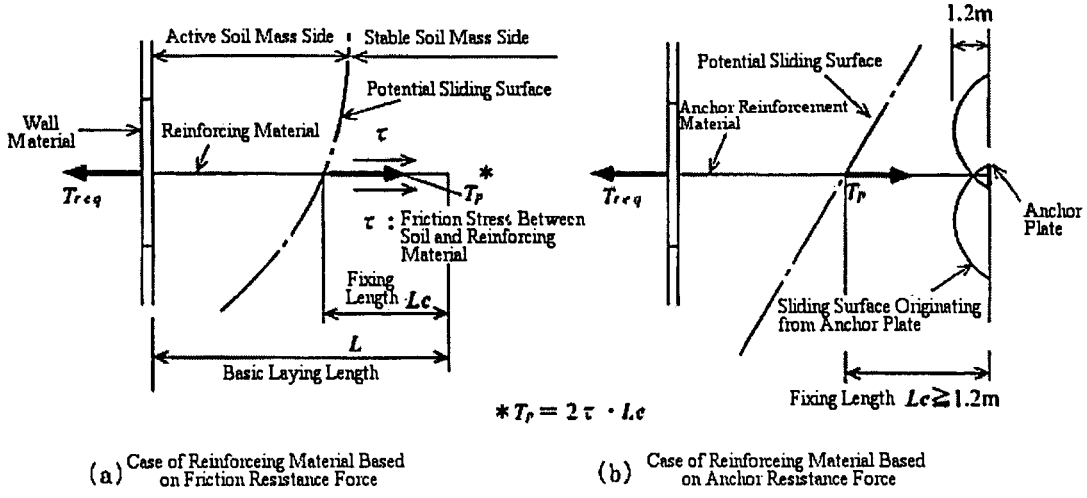


Fig. 3-10-15 Idea of Calculating Pull-out Resistance Force and Deciding Laying Length

2) Stability of Reinforced Area

The application of multiple layers of a reinforcing material generates the effect of integrating the reinforced area and it is necessary to analyse the stability of the integrated area. One method to analyse the stability of the integrated area is to assume that such reinforced area forms a kind of retaining wall and to analyse its sliding, over-turning and bearing capacity as in the case of an ordinary gravity retaining wall.

Under certain circumstances, it is also necessary to analyse the settlement of soft ground as well as the stability against circular sliding, including the foundation ground and the embankment at the back.

3.11 Culvert

A culvert is a structure which is introduced beneath a road, as a water channel for irrigation or drainage purposes or as a passageway for pedestrians or cars. There are many structural types, including box culverts, pipe culverts and corrugated metal culverts.

3.11.1 Selection of Structural Type

Culverts are primarily classified as rigid culverts and flexible culverts. Rigid culverts are designed to use their own rigidity to resist the earth pressure, etc. Box culverts, portal culverts, arch culverts and pipe culverts belong to the category of rigid culverts. Meanwhile, flexible culverts have flexible thin walls. Deflection caused by the vertical earth pressure compresses the soil on both sides of the culvert. Using the reactive earth pressure generated by this compression, this type of culvert resists the external pressure acting on it by equalising such forces along its full arc using the passive earth pressure generated by compression of the soil. Corrugated metal culverts and PVC pipe culverts belong to the category of flexible culverts.

The most suitable structural type must be selected by analysing all of the relevant factors, including the inner dimensions, topography, geology, working conditions and adjacent structures at the proposed site and construction cost. Table 3-11-1 shows the general scope of culvert application.

Table 3-11-1 General Scope of Culvert Application

Type of Culvert	Applicable Overburden	Size of Cross-Sectional Area
Box Culvert [1)]	Up to 10 m	1 x 1 – 5 x 6.5 m
Portal Culvert	Up to 10 m	Span: 3 – 8 m
Arch Culvert [2)]	10 m or more	Span: 3 – 8 m
RC Pipe Culvert	Up to 20 m	150 – 3,000 mm
Prestressed Concrete Pipe Culvert	Up to 30 m	500 – 2,000 mm
Corrugated Metal Culvert	0.6 – 30 m	300 – 4,500 mm
PVC Pipe Culvert (in the case of VU) [3)]	Up to 4 m	150 – 800 mm

Notes

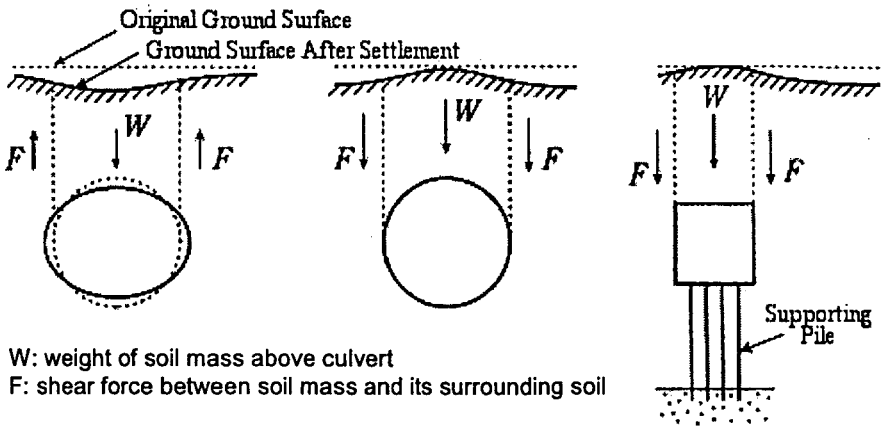
1) The subject here is a cast-in-place concrete box culvert. In the case of precast products, such as PC box culverts, the relevant standards are referred to in order to determine their scope of application.
2) Careful examination is required prior to selection.
3) Two types of PCV pipe culverts, i.e. VP (ordinary pipe) and VU (thin wall pipe) exist. VU is mainly used.

3.11.2 Loads Used for Design of Culvert

The loads used for the design of a culvert include the dead load, earth pressure, live load, impact load and water pressure, etc. It must be noted that the relevance of the loads varies depending on the type of culvert and purpose of design. The main loads are described below. For the design of a culvert, their combination to the best advantage for the purpose of the culvert under the site conditions is essential.

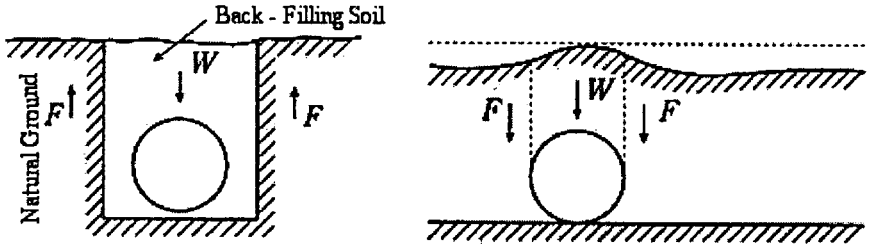
(1) Vertical Earth Pressure

The vertical earth pressure acting on a culvert is basically given as $P_{vd} = \gamma \cdot h$ (γ : unit volume weight of soil; h : thickness of earth cover). However, when a box culvert supported by pile foundations is introduced above soft ground, the settlement of the culvert itself is prevented despite the settlement of the embankment. Here, the acting vertical earth pressure is given as $\alpha \cdot \gamma \cdot h$ as the additional coefficient α is also applicable (Fig. 3-11-1). As such, the vertical earth pressure acting on a culvert varies depending on the bearing conditions of the culvert in addition to the type and burying method, making it necessary to consider an earth pressure suitable for the site conditions (Fig. 3-11-2).



(a) Filexible Culvert (b) Rigid Culvert (c) Supported by Pile

Fig. 3-11-1 Change of Vertical Earth Pressure by Deformation of Culvert and Difference in Settlement Characteristics



(a) Groove Type (b) Projection Type

Fig. 3-11-2 Change of Vertical Earth Pressure by Difference of Laying Method

(2) Horizontal Earth Pressure

The horizontal earth pressure acting on a culvert is given as $P_{hi} = K_0 \cdot \gamma \cdot h_i$ (K_0: coefficient of earth pressure at rest, γ: unit volume weight of soil; h_i: depth at the point in question). However, the coefficient of earth pressure at rest and the distribution of earth pressure vary depending on the type of culvert. The value of this coefficient of earth pressure at rest normally used is 0.5 for a box culvert and portal culvert and 0.3 for an arch culvert.

The horizontal earth pressure is not normally considered for a pipe culvert as it constitutes a safety margin.

(3) Vertical Load Originating from Live Load

The vertical load originating from the live load acting on the upper face of a culvert is generally calculated by making the rear wheel load ($P = 100$ kN (10 tf)) act in the manner shown in Fig. 3-11-3. In this case, the load per unit length along the longitudinal direction of a culvert is calculated by the following formula taking

the impact force into consideration.

$$\text{Rear wheel}: \quad P_{l1} = \frac{2 \times \textit{rear wheel load}\ (100kN(10tf))}{\textit{vehicle width}\ (2.75m)} \times (1+i) \quad \cdots\cdots (3.11.1)$$

The value of the impact coefficient i is normally 0.3 for a box culvert of which the earth cover thickness is less than 4 m. In the case of a pipe culvert, a suitable constant is determined in correspondence with the earth cover thickness. The vertical load on a box culvert becomes smaller as the earth cover thickness increases from 4 m. It is, therefore, sufficient to use the single value of 10 kN/m^2 (1 tf/m^2) for all box culverts with an earth cover thickness of more than 4 m.

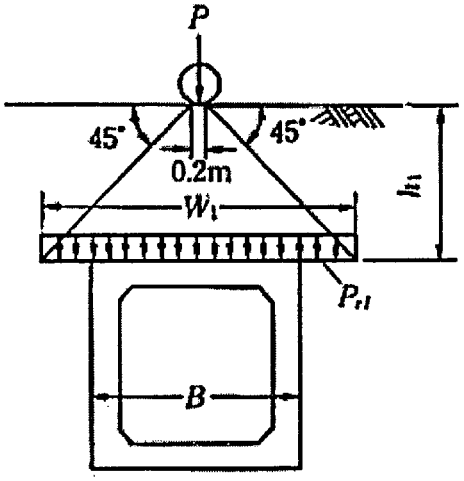


Fig. 3-11-3 Distribution of Live Load

(4) Horizontal Load Originating from Live Load

As the value of the horizontal load originating from the live load acting on both sides of a culvert, $10K_0$ kN/m (surcharge 10 × coefficient of earth pressure at rest K_0) is given to each side. It is a common practice not to consider the horizontal load for pipe culverts.

(5) Effects of Earthquakes

As relatively small underground structures such as culverts move in unison with deformation of the surrounding ground or embankment at the time of an earthquake, the bending and shearing forces acting on the body due to the effects of an earthquake are believed to be small.

Accordingly, it is unnecessary to consider the effects of earthquakes for ordinary culverts. However, consideration of the effects of earthquakes for large-scale portal culverts is recommended.

3.11.3 Important Points for Design of Various Types of Culverts

A culvert which has the required strength and durability and which is economical to construct and appropriate for the construction site must be designed based on its purpose(s) and the topographical, geological and working conditions of the site. When designing a culvert, therefore, it is essential to pay proper attention to the characteristics and design principles of each type of culvert described below.

(1) Box Culvert

A box culvert is commonly constructed using cast-in-place reinforced concrete and is used as a road or water channel. In the case of a box culvert which is used as a road, the most common section is a single section which must be determined by analysing the width of the existing road, through traffic, road alignment, presence of a water channel, etc. and future plans for the site. A single section is desirable for a box culvert used as a water channel. Dual or multiple sections may be adopted, however, if restrictions posed by the inner height of the culvert make it impossible to secure the necessary cross-sectional area.

Fig. 3-11-4 shows the common configuration of the members and design of a box culvert to be introduced beneath a road.

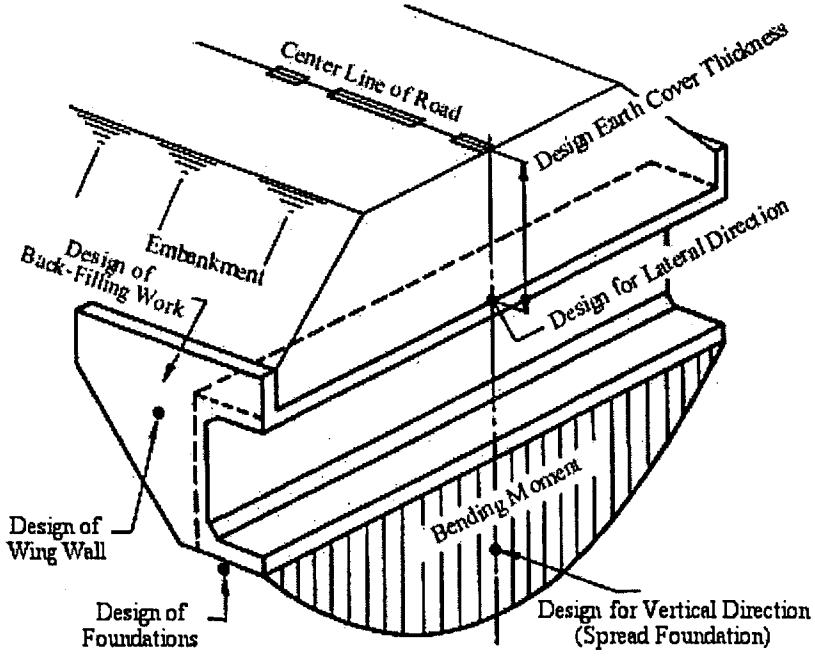


Fig. 3-11-4 Members and Design of Box Culvert

The earth cover thickness is naturally determined by the required cross-sectional area of the box culvert, embankment height and elevation of the box culvert. In the case of shallow earth cover, as the road surface

often becomes uneven due to settlement of the back-filling material, a design earth cover thickness of not less than 50 cm is desirable.

A box culvert must be designed in both the cross-sectional and longitudinal direction as shown in Fig. 3-11-4. As a rigid-frame axis for calculation along the cross-sectional direction (span direction), the central axis dimension lines (Bo, Ho) shown in Fig. 3-11-5 are used. In principle, expansion joints are introduced at intervals of 10 – 15 m along the longitudinal direction of a box culvert regardless of the foundation conditions. When the interval of the expansion joints is determined together with the application of distributing bars (transverse reinforcement) corresponding to the main reinforcement for the cross-sectional direction, the design for the vertical direction can generally be omitted. Nevertheless, analysis for the vertical direction is required when spread foundations are employed above poor foundation ground or when the length exceeds 15 m. In principle, calculation for the vertical direction assumes "beams above an elastic foundation".

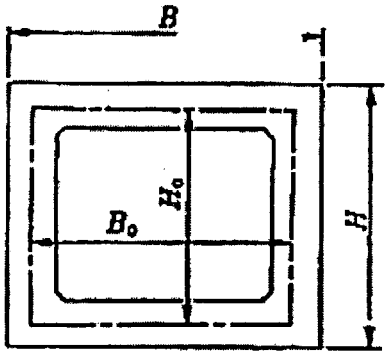


Fig. 3-11-5 Rigid Frame Axis

(2) Portal Culvert

A portal culvert has a structure where footings are introduced at the bottom of the side wall without a bottom plate. A portal culvert is generally constructed at sites with good foundation ground and is opted for when the inner cross-sectional area is large or when the construction of a bottom plate for a box culvert is difficult because of the site conditions, including the impossibility of unwatering.

The basic design requirements are similar to those for a box culvert although design which considers the effects of temperature fluctuation and drying shrinkage is necessary when the earth cover is shallow or when a long span makes it necessary to treat a portal culvert as a bridge. Moreover, when the scale of a portal culvert is truly large, it is desirable to consider the effects of earthquakes using the earth pressure at the time of an earthquake determined by the Mononobe-Okabe method.

For the design of the body, strats should, in principle, be applied between the footings to prevent the destruction of the corner sections by sliding of the footings. In the case of the foundation ground being soft rock or harder, these strats may be omitted if sliding of the footings is prevented by some means.

(3) Arch Culvert

An arch culvert receives vertical earth pressure with its arching member and is generally more economical than a box culvert under the condition of a high embankment. Careful analysis must be conducted prior to its selection as there are conditions for its use. These conditions are that differential settlement due to inclined ground does not occur and that unbalanced earth pressure due to the topography and/or different banking materials used does not occur.

The arching member must have a thickness with a margin which ensures its safety when exposed to load during construction work or unbalanced earth pressure. It is, therefore, desirable to determine the thickness by considering its balance with the side wall members. A member thickness of some 60 cm or more has often been adopted by past examples (Fig. 3-11-6).

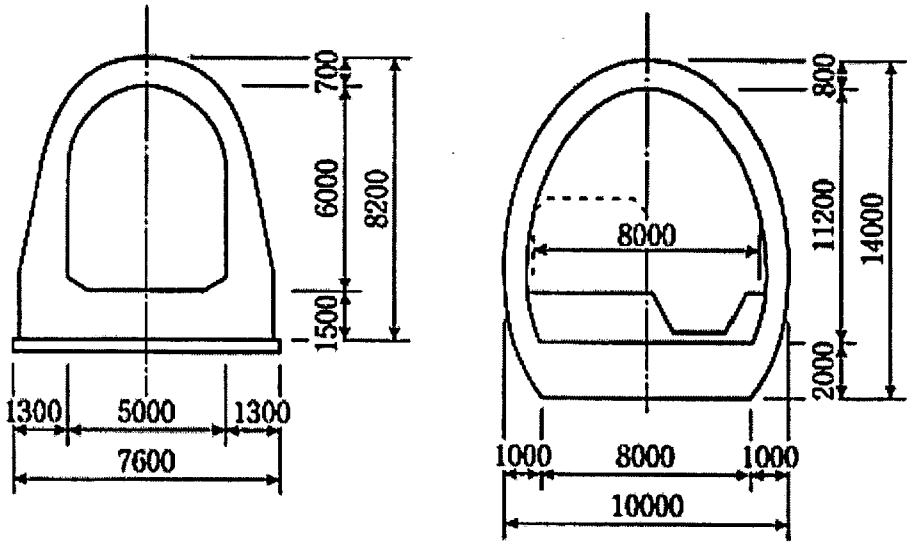


Fig. 3-11-6 Typical Shape of Arch Culvert

(4) Rigid Pipe Culvert

Pipe culverts are widely used as a permanent or temporary structure to be used as a water channel. In general, a RC pipe (JIS A5303) or prestressed concrete pipe (JIS A5333) is used to act as a concrete pipe culvert. The most popular pipe is a centrifugal RC pipe while a prestressed concrete pipe is used at sites with a thick earth cover.

For the design of a pipe culvert, the pipe diameter and burying method are first determined given the fact that the body is a precast product. The type of foundations and the type of pipe corresponding to the required pipe culvert strength are then selected.

There are two pipe culvert burying types, i.e. the projection type and the groove type, as shown in Fig. 3-11-7. As each type uses a different earth pressure concept, attention should be paid to such a difference at the design stage. The projection type means that the pipe is laid directly onto the ground or well-compacted ground and is banked afterwards as shown in Fig. 3-11-7-(a). In contrast, the groove type means that a groove is dug in the original ground or well compacted ground to bury the pipe as shown in Fig. 3-11-7-(b). A pipe culvert which is buried after excavation of a pre-loaded and rested embankment for a long period of time is considered to be the groove type. However, pipe culverts under the conditions shown in Fig. 3-11-8 must be considered the projection type.

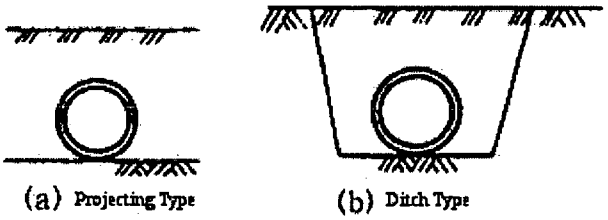


Fig. 3-11-7 Laying Method

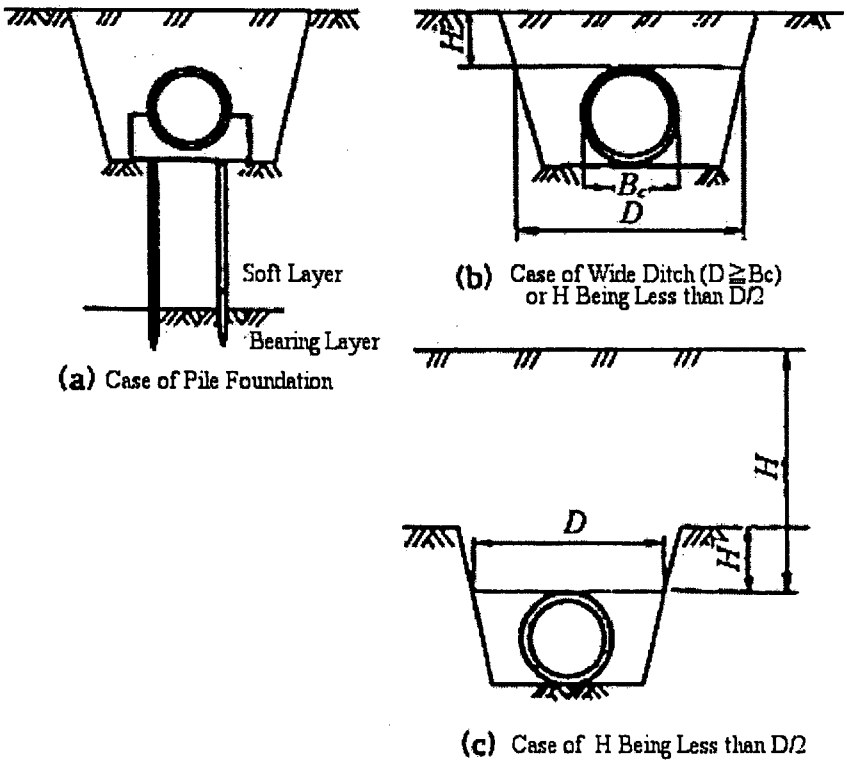


Fig. 3-11-8 Conditions considered as Projection Type

The standard types of foundations for rigid pipe culverts are sand foundations, crusher run foundations and concrete foundations. In general, sand foundations or crusher run foundations are used in the case of relatively good ground while concrete foundations are used when large external forces act on the pipe and the foundation ground is soft.

(5) Corrugated Metal Culvert

Corrugated metal culverts are used at sites where their characteristic flexibility can be utilised. As the light weight corrugated section is advantageous for transportation and there is little demand for large construction machinery, corrugated metal culverts are often used at remote mountainous sites. Corrugated metal culverts are also often used as temporary water channels because of the ease of site work as well as conversion to other uses.

There are four types of sectional forms, i.e. circular, elongation, pipe arch and arch. Among these, the circular form is most popularly used because of its mechanical stability and ease of assembly and construction compared to other forms. In the case of the elongation form, inverse deformation of 5% is given along the vertical direction in advance and it is used for high embankments where the circular form exceeds the limit value for flexibility. The pipe arch form is mainly used when the thickness of the earth cover is small while the arch form is used when a large clearance limit is required in the case of a footpath under a road, a vehicle road or a water channel requiring a large cross-sectional area of flow.

Design for the use of a corrugated metal culvert is required to calculate the cross-sectional rigidity during construction work, axial joint strength, buckling of the corrugated section and deflection of the corrugated metal culvert so that the calculated values satisfy the relevant allowance values.

The design of the foundations, selection of the back-filling material and acting of unbalanced earth pressure, etc. are also important to ensure the full performance of the characteristics of a corrugated metal culvert and proper attention should be paid to these aspects in the design of a corrugated metal culvert.

(6) PVC Pipe Culvert

There are four types of PVC pipe culverts, i.e. round pipe, oval pipe, high rigidity round pipe and high rigidity oval pipe. Given its light weight, the unit length of PVC pipe can be quite long. In addition, PVC pipe is highly resistant to both acid and alkali. The use of oval pipe is often associated with low water

discharge or a gentle gradient.

Design for the use of PVC pipe is required to calculate the maximum bending stress and rate of deflection of the pipe so that the calculated values satisfy the relevant allowance values.

3.11.4 Design of Foundations

The foundations for a culvert can be spread foundations using good quality soil, such as sand, replacement foundations or pile foundations, etc. as shown in Table 3-11-2 depending on the actual ground conditions. For the design of foundations, the following points must be noted.

① In principle, the foundations for a culvert should be spread foundations. When pile foundations of which the structure prevents any settlement of the culvert are required to support a water channel culvert on soft ground, the possibility of an increased surcharge due to settlement of the surrounding ground and an uneven road surface must be carefully analysed followed by the preparation of suitable countermeasures.

② When a culvert is constructed on soft ground, the ground should be improved by the surcharge method.

③ When a thin soft layer is situated near the ground surface or when the ground partially consists of a soft layer, the soft soil should be replaced by good quality soil.

Table 3-11-2 Criteria for Selection of Foundations

Ground	Ordinary Ground	Soft Ground
Type of Foundations	Spread Foundations	▪ Replacement Foundations ▪ Ladder Type Foundations ▪ Sand Sheet Foundations ▪ Soil Cement Foundations ▪ Pile Foundations

Note: Foundations combining different types of foundations

4. Execution

4.1 Introduction

The basic principle of execution is the construction of a road of exactly the shape and quality specified in the design documents while ensuring its conformity with the local topography and geology. The actual execution processes of the work are subject, however, to natural phenomena, including the weather conditions, and there are many factors which may disrupt execution of the work. As road earthwork is closely related to the social environment in the surrounding area, this work cannot be executed without considering such environment. It is, therefore, essential to properly recognise possible factors of disruption and the social environment so that the work can be smoothly conducted.

Special attention must be paid to the following general requirements.

① Proper understanding of the contents of the design documents and site conditions and precise control of work execution

② Precise knowledge of the local geology and soil properties and careful attention to rainwater and groundwater

③ Conscious efforts to ensure safe execution, disaster prevention and conservation of the surrounding environment

④ Constant smooth communication between the owner and the contractor

4.2 Execution Plan

For the economical, speedy, precise and safe execution of road earthwork to construct an embankment or structure while achieving the required quality, it is essential to prepare an appropriate execution plan prior to execution by fully understanding the contents of the work and the work conditions based on a relevant study.

4.2.1 Preparation of Execution Plan

The normal procedure for the preparation of an execution plan is the preparation of an overall plan to ensure the completion of the work within a specified work period, followed by the adjustment of various conditions which pose bottlenecks for the progress of the work.

An execution plan incorporates all of the relevant matters for work execution and its main items are listed below.

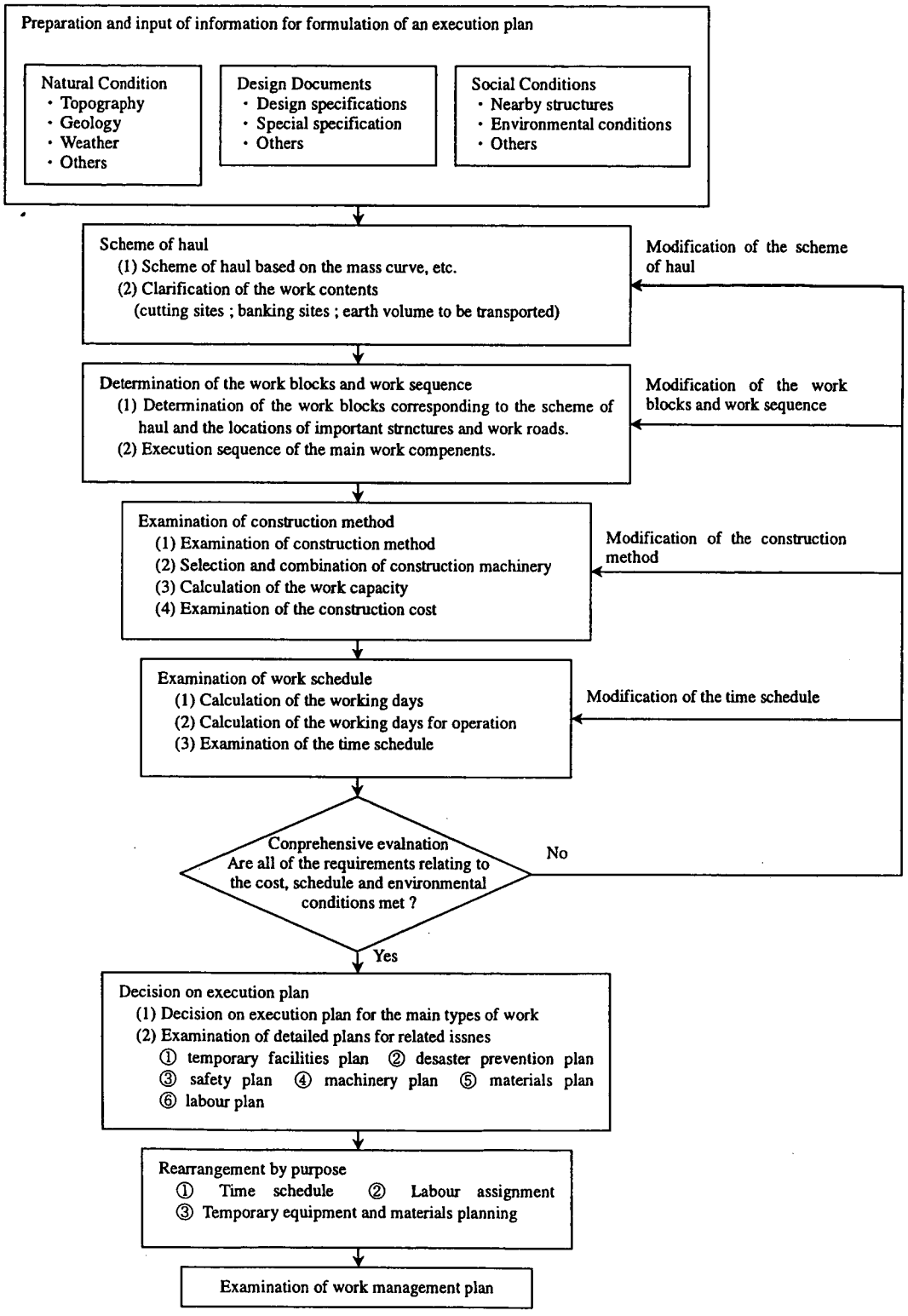


Fig. 4-2-1 Flow of Preparation of Execution Plan

① Plan for scheme of haul

② Construction method, plan for use of required construction machinery, work execution speed and required time for each type of work

③ Order and timing of execution for each type of work and the overall schedule

④ Labour plan and equipment and materials plan

⑤ On-site execution system and temporary facilities plan

⑥ Plan for construction roads and other preparatory work

⑦ Plan for accident prevention, safety and hygiene

⑧ Plan for conservation of the surrounding environment

Fig. 4-2-1 is a flow chart which shows the planning processes of an earthwork execution plan.

4.2.2 Scheme of Haul

The scheme of haul is a central issue in an earthwork execution plan. Firstly, the basic soil properties are established based on soil and geological survey data and site reconnaissance results. Meanwhile, the earth volume is calculated using profile and cross-sectional drawings and is arranged by place, block, soil type and purpose of use. A more accurate earth volume can be calculated by multiplying each type of soil by the relevant bulking factor of soil. .

The bulking factor of soil, L and C are defined by the formulas given below. Table 4-2-1 shows the average bulking factor for each type of soil based on past data.

$$L = loose\ earth\ volume\ (m^3) / original\ earth\ volume\ of\ ground\ (m^3)$$

$$C = compacted\ earth\ volume\ (m^3) / original\ earth\ volume\ of\ ground\ (m^3)$$

Table 4-2-1 Bulking Factor of Soil

		L	C
Rock or Stone	Hard rock	1.65 – 2.00	1.30 – 1.50
	Medium hard rock	1.50 – 1.70	1.20 – 1.40
	Soft rock	1.30 – 1.70	1.00 – 1.30
	Rock mass/boulders	1.10 – 1.20	0.95 – 1.05
Soil with Gravel	Gravel	1.10 – 1.20	0.85 – 1.05
	Gravelly soil	1.10 – 1.30	0.85 – 1.00
	Solidified gravelly soil	1.25 – 1.45	1.10 – 1.30
Sand	Sand	1.10 – 1.20	0.85 – 0.95
	Sand with rock mass/boulders	1.15 – 1.20	0.90 – 1.00
Ordinary Soil	Sandy soil	1.20 – 1.30	0.85 – 0.95
	Sandy soil with rock mass/boulders	1.40 – 1.45	0.90 – 1.00
Cohesive Soil, etc.	Cohesive soil	1.20 – 1.45	0.85 – 0.95
	Cohesive soil with gravel	1.30 – 1.40	0.90 – 1.00
	Cohesive soil with rock mass/boulders	1.40 – 1.45	0.90 – 1.00

The next step is the preparation of an earth volume calculation sheet and a mass diagramme to plan the minimum sum of "the earth volume to be transported ×the transportation distance" under given conditions. For this purpose,

careful attention must be paid to the efficient locationing of the borrow pit, spoil area and temporary storage yard. Fig. 4-2-2 shows an example of a haul scheme diagramme.

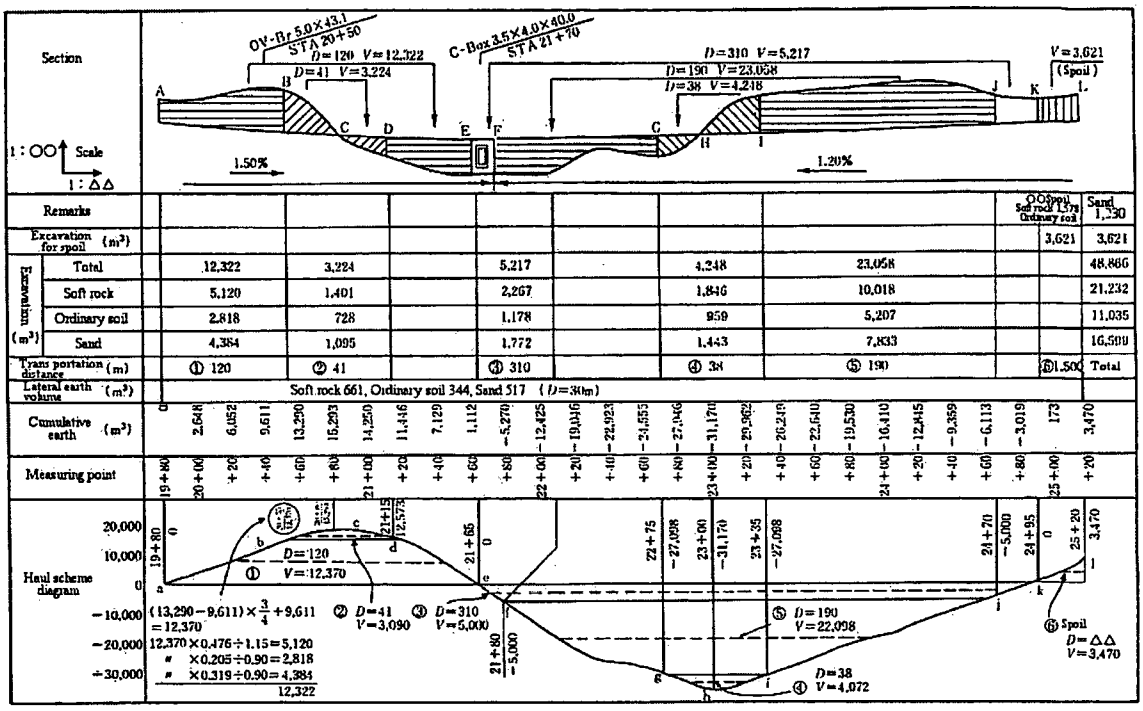


Fig. 4-2-2 Distribution of Earth Volume Based on Haul Schedule Diagram

4.2.3 Excavation and Transportation

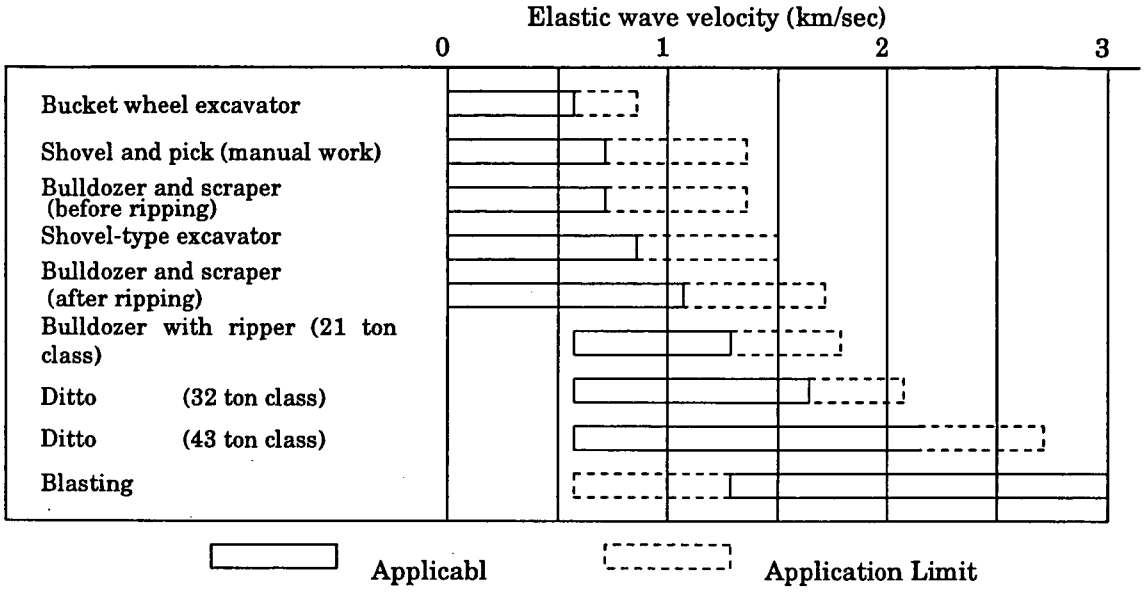
For excavation work, the most suitable method and machinery to be used should be selected based on the site topography, excavation height, excavation volume, state of the stratum (presence of rocks, etc.) and viable hauling method for the excavated soil. When the soil to be excavated is hard, it is useful to establish the excavation limit for the excavators in advance using the elastic wave velocity as shown in Table 4-2-2.

The basic excavation methods are bench cutting and downhill cutting as shown in Fig. 4-2-3. The former is suitable for large-scale work and it is necessary to determine the bench height suitable for the planned excavator to be used. A bulldozer, scrapedozer and/or scraper are used in the case of the latter to excavate and haul the soil utilising a downward slope.

There are many types of transportation machinery with different structures and functions, including bulldozers, scrapers and dump trucks. While caterpillar (crawler)-type machinery can overcome the ground undulations at an earthwork site to proceed with the work, wheel-type machinery requires the availability of a certain travelling surface to ensure its effective use. The latter, therefore, requires haulage roads on site. As the work efficiency is

considerably affected by the quality of such roads, it is important to select the correct machinery and its proper use to suit the specific site conditions.

Table 4-2-2 Application Limit of Excavation Method



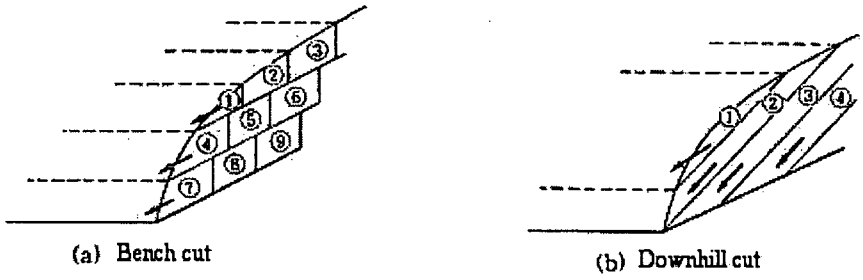


Fig. 4-2-3 Excavation Methods

4.3 Banking Work

4.3.1 Basics of Banking Work

The basic execution processes for road banking work in accordance with an execution plan are shown in Fig. 4-3-1.

The important points to be considered in relation to banking work include the execution of stabilisation work, banking using rock mass or volcanic cohesive soil as the banking material, banking when a pipe culvert crosses a road, deformation of the surrounding ground by an embankment and its impacts, execution speed of banking, measures to ensure trafficability for construction machinery and environmental and disaster prevention measures.

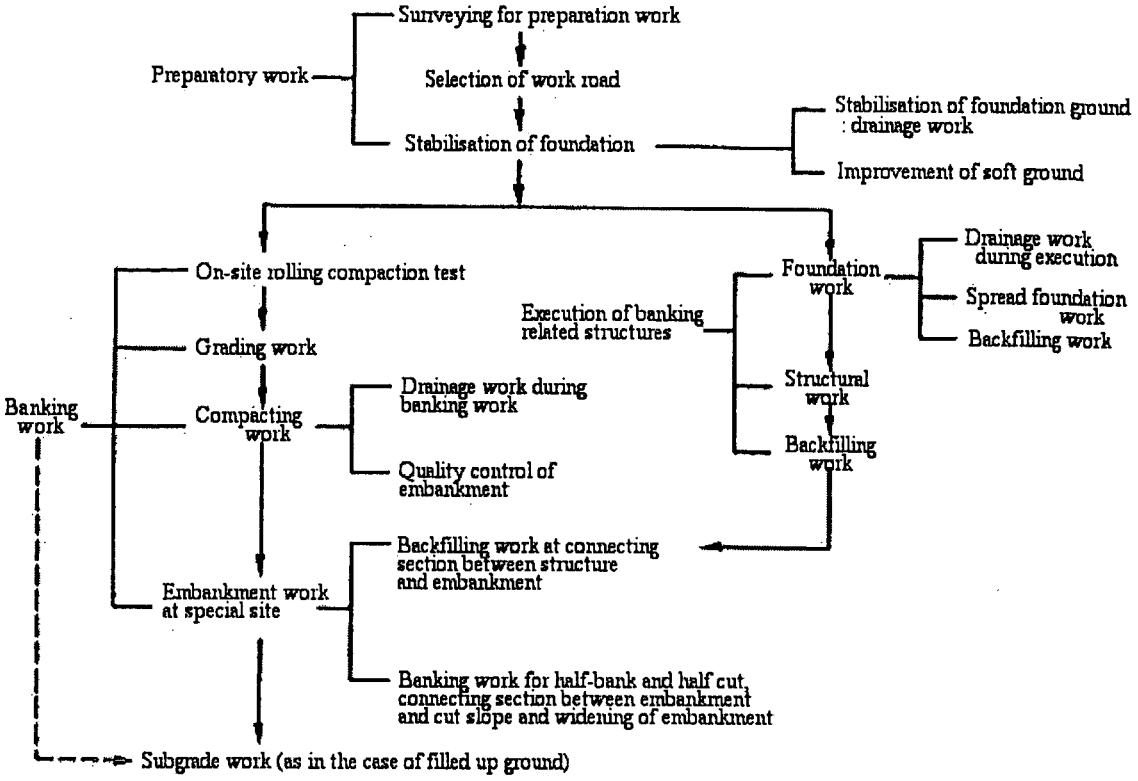


Fig. 4-3-1 Sequence of Banking Work

An embankment is generally liable to adverse effects by water and has such destabilising factors as softening, settlement and failure which are unique to soil. However, these problems can be overcome with the proper selection and improvement of the banking material and the employment of a suitable execution method for the construction of an embankment as a homogenous and highly stable structure.

An embankment is required to have the following two types of performance.

① As the foundation for the pavement, an embankment must bear the traffic load without fail and its deformation and/or settlement after completion must not disrupt vehicle traffic.

② An embankment must be highly resistant to such causes of failure as wind/flood damage and earthquakes, etc.

4.3.2 Treatment of Foundation Ground

Treatment of the foundation ground prior to banking mainly has the following purposes.

① Smooth integration of the embankment and foundation ground

② Smoothing of the initial banking work

③ Stabilisation of the ground to increase its bearing capacity

④ Prevention of settlement caused by the decay of harmful objects, such as plants and trees

(1) Treatment of Ordinary Ground

While no special foundation stabilisation work is necessary for ordinary ground, the work listed in Table 4-3-1 must be conducted without fail.

Table 4-3-1 Treatment of Ordinary Foundation Ground

Work Item	Purpose	Guidelines or Work Contents
Clearing and Grubbing	Prevention of loosening of embankment or harmful settlement due to decay of plants after banking	(1) Clearance of trees at a point near the original ground surface (2) Removal of stumps, bamboo roots and other obstacles within 1 m below the design subgrade plane (3) Removal of deposited leaves or dead branches at dipped sites in a mountain area (4) Removal of grass, trees, stumps and bamboo roots prior to excavation at a borrow pit
Top Soil Stabilisation	Prevention of adverse impacts on embankment	When top soil consisting of humus soil, etc. forms part of the subgrade for an embankment, it is necessary to excavate it to the required depth for replacement by a banking material.
Stabilisation of Thin Soft Layer	Securing of trafficability and sufficient compaction of banking material	(1) Introduction of a drainage ditch(es) at the foundation ground for an embankment to dry the said foundation ground; filling of such a drainage ditch(es) with sand and/or crusher-run to make it function as a drainage ditch(es) for the embankment site (2) Introduction of a sand mat (sand layer) with a thickness of some 0.5 m (3) Drainage through a subsurface drainage ditch(es) or drainage pipe(s) when a minor pond or spring water exist at an embankment site
Stabilisation of Faulting	Sufficient compaction of embankment and homogenisation of embankment	When the foundation ground of an embankment is extremely uneven or has faulting, levelling work should be conducted prior to banking to make the ground as flat as possible.

(2) Treatment of Inclined Ground

As such problems as differential settlement and sliding of the embankment tend to occur with banking on inclined ground, it is necessary to pay careful attention to its execution as in the case of the half bank, half cut work described in 4.3.8-(1).

4.3.3 Banking Materials

As the types of soil to be used for specific parts of an embankment are roughly decided at the design stage, the main focus at the execution stage should be placed on a good understanding of their properties to ensure excellent execution. When there is room in the selection of the material, it is desirable to select a good quality material

which is easy to work with, offers high strength after compaction, has low compressibility, is highly resistant to inundation by rainwater and has low swelling after water absorption.

At the execution stage, careful attention should be paid to the following points.

i) Soil containing a large amount of decayed grass, trees and/or stumps, etc. should not be used.

ii) Even if the banking material is designated at the design stage, a countermeasure, such as soil quality improvement or converted use, should be considered in the case of the designated material having high compressibility as well as insufficient shear strength at the execution stage in view of the height and shape of the embankment.

iii) When the trafficability for construction machinery is insufficient, the introduction of a countermeasure(s) should be considered.

iv) Although it is difficult to conduct placing, spreading and compacting work with soil which has a high mixture ratio of crushed rock, rock mass and boulders, this type of soil offers a high level of stability when the embankment is completed.

v) When soft rock muck produced by tunnel excavation, etc. is used as the banking material, it may easily become soft due to kneading by machine under the condition of slaking. It is, therefore, essential to judge the rock properties in advance.

4.3.4 Stabilization

When it is necessary to use cohesive soil with high water content or a material of which the strength may be insufficient, stabilisation is conducted by mixing the soil with a stabiliser, such as cement or lime. There are several stabiliser mixing methods and the effect of stabilisation work is determined by the quality of mixing.

The principal mixing methods are the central plant mixing method and the mixed-in-place method. The former is suitable for materials with a low viscosity and is used when the work volume is large. The latter is suitable for lime stabilisation work which often uses a banking material with high water content. There are two ways of conducting mixed-in-place method: (i) agitated mixing with a stabiliser and a backhoe and (ii) cut mixing by a grader. What is important with mixed-in-place method is the uniform spraying of the stabiliser.

When using quick lime, mixed-in-place method is usually conducted twice. There are also other methods, including the admixture of a stabiliser when the banking material is loaded onto a truck, etc. and the excavation and loading of soil after a stabiliser has been injected into the ground in a pile fashion.

4.3.5 Spreading and Adjustment of Water Content

(1) Spreading of Ordinary Banking Material

Placing and spreading is the most important work in the construction of a uniform embankment. As the thickness of spread materials is affected by such conditions as the particle size and properties of the banking soil, type of compaction machine, execution method and required degree of compaction, it is preferable to decide the thickness through trial execution. The spreading thickness for each layer should be determined based on the finished thickness after compaction specified in the specifications. In general, the spreading thickness for a single layer is up to 35 – 45 cm for filled up ground (compacted thickness: 30 cm) and up to 25 – 30 cm for the subgrade (compacted thickness: 20 cm).

(2) Spreading of Banking Material with High Water Content

When cohesive soil with high water content is used as the banking material, its strength can substantially decline due to rutting and kneading by transportation machinery. Therefore, the direct delivery of such soil onto an embankment by a scraper, shovel loader or dump truck is not feasible. This makes it necessary to conduct the secondary haulage of the banking material from an unloading site on the embankment to the banking site for placing and spreading. While this secondary haulage is conducted as part of the placing and spreading, the distance to push the soil is longer than usual. For this secondary haulage, a swamp bulldozer with a low contact pressure is used. The haulage road for a dump truck is generally rerouted upwardly in line with the progress of banking work (see Fig. 4-3-2).

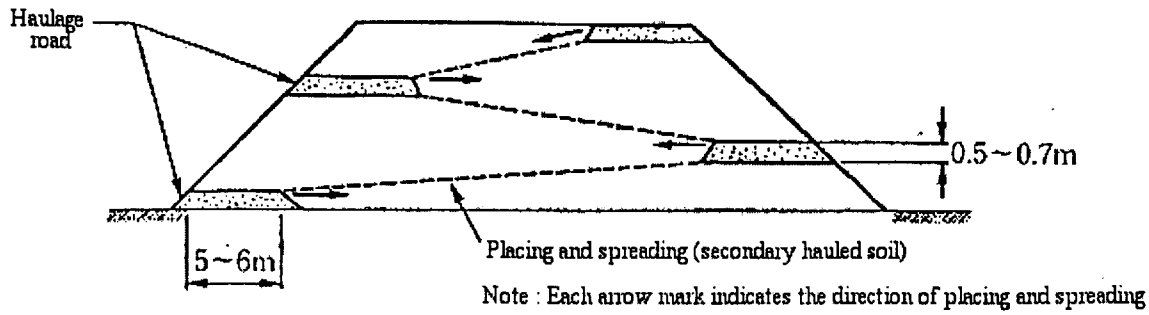


Fig. 4-3-2 Example of Placing and Spreading of Banking Materials with High Water Content

When the construction of a high embankment using cohesive soil with high water content is planned, several horizontal drainage layers made of pit sand with excellent permeability or a draining geotextile are introduced at certain intervals for the purpose of dewatering of the embankment through these drainage layers using porous pipes, etc.

(3) Adjustment of Water Content

When the natural water content of the banking material does not fall within the water content range during construction set forth for compaction work, it is necessary to adjust the water content so that it falls within the said range. This adjustment can be conducted by (i) aeration drying to reduce the water content, (ii) excavation of a trench to reduce the water content of the banking material at the cutting site or borrow pit or (iii) spraying to increase the degree of compaction of dry sand or dry clay.

Aeration involves the placing and spreading of the material prior to compaction so that the soil is left alone and then raking using a harrow. Aeration has many disadvantages, including its requirement of a large area and decline of the work speed. In contrast, the excavation of a trench is intended to lower the water content of the material by lowering the groundwater level at a borrow pit and its relative effectiveness is accepted.

(4) Placing and Spreading of Rock Masses

When rock masses are used for banking purposes, they may be used as they are or may be broken into smaller pieces.

1) Large Rock Masses or Boulders

Large rock masses or boulders are used for the lower section of an embankment or at the bottom of the slope. They are placed and spread by a bulldozer or backhoe. The gaps between them should be carefully filled with soil to minimise such gaps. In the case of a gently sloping surface, rock masses or boulders may be spread all over the slope surface.

2) Medium Size Rock Masses or Boulders

Rock masses or boulders with a diameter of approximately up to 50 cm should be used at the lower part of filled up ground. They should be spread by a bulldozer or backhoe and the gaps should be filled with soil. As the spreading thickness of soil containing rock masses or boulders generally becomes 1.5 – 2.5 times the rock mass or boulder diameter, a large vibration roller or similar should be used for compaction to ensure the filling in of soil into the gaps between the rock masses or boulders.

3) Small Rock Masses or Boulders

In the case of rock masses or boulders with a diameter of approximately 10 – 20 cm, they should be mixed with a sufficient amount of soil to fill in the gaps or should be placed and spread in a thin sandwich manner for compaction by a vibration roller or similar.

4) Mudstone

In the case of mudstone which is liable to slaking, it should be placed and spread in a sandwich manner together with sandy soil, if possible, and should be firmly compacted by a large compactor to prevent any settlement in the future. As mudstone tends to shrink, swell, exfoliate and pulverise due to repeated drying and wetting through contact with air, it should be placed in the central part of an embankment to avoid the adverse impacts of repeated drying and wetting.

4.3.6 Compaction

Compaction is the most important step of execution as it directly determined the embankment quality. Compaction has the following purposes.

i) To achieve the most stable state of the soil by curtailing softening and swelling of the soil, in turn resulting from the lowered permeability of the soil due to reduced voids in the soil
ii) To give an earth structure the required strength characteristics in the form of a stable embankment slope and sufficient bearing capacity
iii) To reduce the compressive settlement of the embankment itself after completion

For the compaction of an embankment, an appropriate method must be employed to achieve the qualitative targets in terms of the degree of compaction, water content during construction and spreading thickness of the banking material as stipulated in the specifications.

(1) Compacting Work and Compacting Equipment

For the execution of compacting work, it is necessary to select appropriate equipment and to secure the construction of an embankment of the required quality in accordance with the spreading thickness, number of compaction operations and water content during construction as identified by trial execution. There are basically two types of compaction work rules to secure the embankment quality: (i) quality-related rules and (ii) compaction method-related rules. The quality-related rules can secure the achievement of the required

quality by means of (i) density comparison, (ii) degree of saturation or air void ratio and (iii) strength characteristics. Meanwhile, the compaction method-related rules set forth the type of equipment used for compaction and number of compaction operations. Proper understanding of the characteristics of each method is essential so that a suitable method is selected by thorough examination of the site conditions, including the character and scale of the intended work and the soil conditions.

The in-situ measuring of the density is conducted by the sand replacement method or the core cutter method. In the case of large-scale earthwork, the method using a radio isotope density gauge is used because of its convenience and quick production of the measuring results. As the method using a radio isotope density gauge does not require a conventional complicated calibration test, new quality control using the characteristics of a radio isotope density gauge have been proposed and put into practice. See Table 2-10-1 for the in-situ density measuring methods.

For the execution of compaction work, careful attention should be paid to the following points.

i) The entire embankment should be evenly compacted. Special attention should be paid to the sides and corners of the embankment as compaction is frequently insufficient in these areas.

ii) During banking, special attention should be paid to drainage by ensuring a cross slope (approximately 4%). When rain is predicted, the embankment surface should be flat by a roller to prevent the occurrence of standing water or the excessive permeation of water into the embankment.

Compacting equipment is selected taking such construction conditions as the properties of the banking soil, type of construction method and scale of work and the characteristics of the compacting equipment into consideration. In particular, the soil properties are a key point.

4.3.7 Subgrade Work

The subgrade is an extremely important part of the road structure as it conveys the traffic load to underneath the subgrade. The qualitative requirements of the subgrade are, therefore, the required bearing capacity, resistance to differential settlement and homogeneity.

(1) Subgrade Work at Cutting Section

At a cutting section, the ground itself can be used as the subgrade provided that it has a sufficient bearing capacity. However, careful attention is required to not damaging the subgrade surface by excavation and transport equipment. When the subgrade section cannot provide the required bearing capacity due to its

softness, replacement by good quality soil or the introduction of stabilisation work is required. When tree roots, boulders and others which may considerably damage the homogeneity and durability of the subgrade are found at a subgrade section, a remedial measure(s), including removal and replacement by good quality soil, is necessary.

(2) Subgrade Work at Banking Section

The desirable subgrade materials are those which are easy to work with and which offer high strength after compaction and low compressibility. The maximum particle size is preferably 80 – 100 mm or less to ensure the homogeneity of the subgrade. When excavated soil is used as a subgrade material, soil of as high a quality as possible should be used. If such high quality soil is unavailable, the use of a borrow material or soil improvement should be considered.

Placing and spreading for the subgrade should be conducted to achieve an each layer thickness of 20 cm or less and each layer must be fully compacted in a homogeneous manner. The finished surface of the subgrade must be flatly compacted by rolling to prevent the permeation or standing of rainwater and should be given an appropriate cross slope. This cross slope should be slightly increased when there is a long period before the commencement of paving work.

(3) Surface Drainage of Subgrade

It is desirable to quickly drain surface water from the road which is produced by rain after compaction of the subgrade by introducing temporary drainage channels at the shoulders.

4.3.8 Execution of Connecting Section between Cut Slope and Embankment

(1) Half Bank, Half Cut Section and Connecting Section between Cutting and Banking

At a half bank, half cut section or a connecting section between cutting and banking, cracks in the pavement often occur after the completion of the work due to faulting caused by (i) discontinued bearing capacity between the cutting section and the banking section, (ii) softening of the embankment because of the accumulation of spring water or seepage water at the boundary between the cutting section and banking section, (iii) insufficient compaction of the embankment at the boundary and/or (iv) sliding because of insufficient adhesion between the inclined foundation ground and the embankment at the half cut, half bank section. Possible countermeasures include bench cutting, smooth transition at the cutting section and

drainage work as shown in Fig. 4-3-3. For subgrade constructed at the boundary between the original ground and embankment, compaction should be conducted after backfilling with the same banking material to avoid a sudden change of the soil quality. Fig. 4-3-4 shows the treatment to be employed at the time of execution.

At a longitudinal connecting section between cutting and banking, a smooth transition section should be introduced as shown in Fig. 4-3-5 for backfilling with the same banking material and compaction thereafter.

When spring water is present at the foundation ground for an embankment, a drainage layer made up of highly permeable material as shown in Fig. 4-3-6 or dry masonry work should be introduced to prevent the accumulation of water inside the embankment. Depending on the amount of spring water, it may be necessary to bury porous pipes.

In the case of half bank, half cut work, thick spreading as shown in Fig. 4-3-7 often occurs as excavated soil from the cutting section is simply moved in the cross-sectional direction. In this case, a bulldozer for spreading should be deployed at the banking site so that excavated soil from above can be quickly spread while avoiding the accumulation of such soil above the compacted embankment.

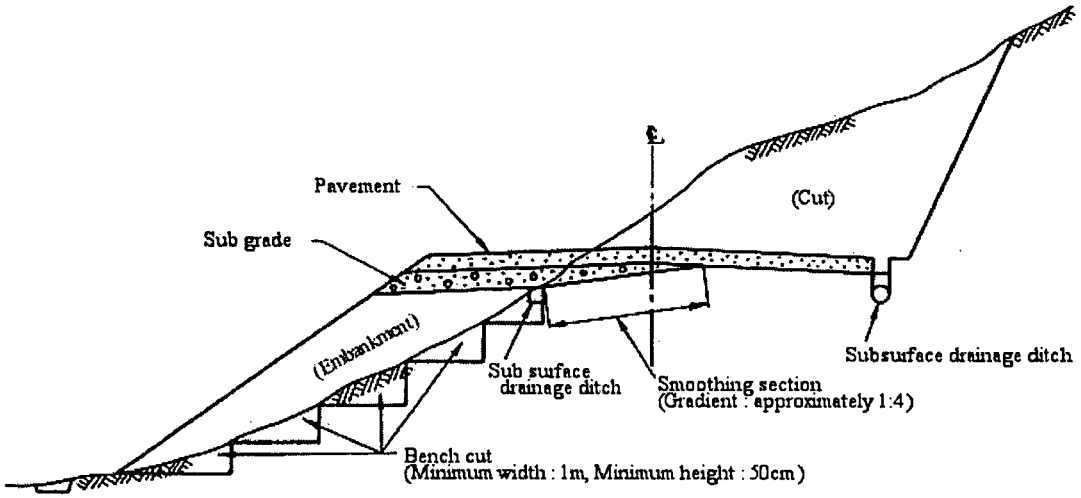


Fig. 4-3-3 Bench Cutting of Foundation Ground for Embankment and Connecting Section Between Cut Slop and Embankment (Example of Sandy Ground)

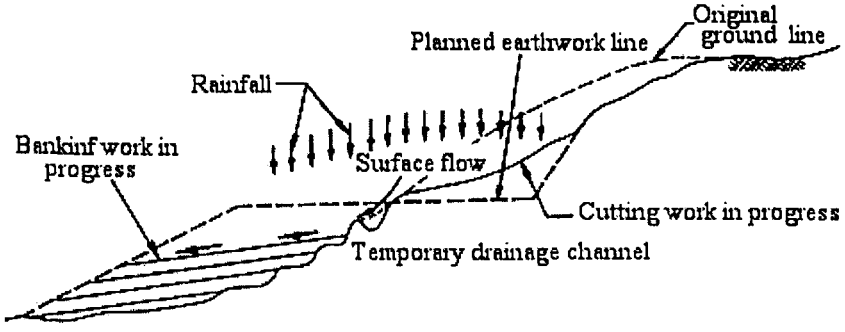
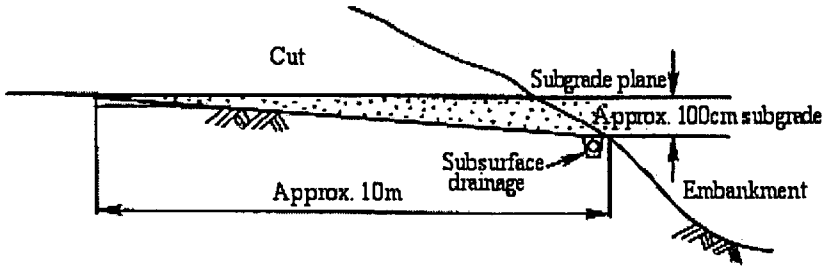
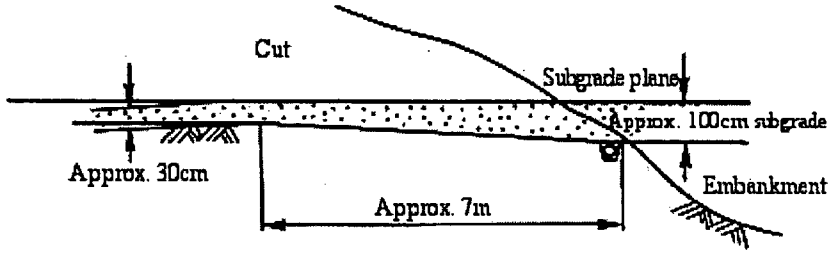


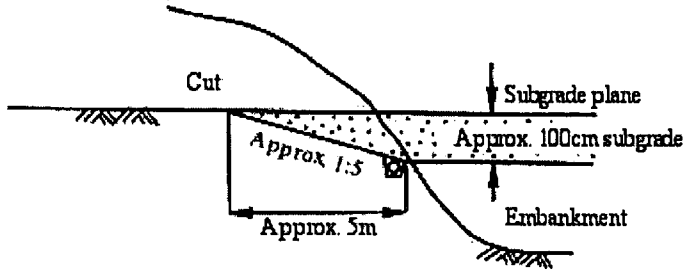
Fig. 4-3-4 Countermeasures for Rainfall during Half-bank and Half cut work



(a) Case with no soil replacement for subgrade at cutting section



(b) Case with soil replacement for subgrade at cutting section



(c) Case where a long smoothing section is uneconomical because of original rock ground

Fig. 4-3-5 Example of Smoothing Work at Boundary between Cutting and Banking Section (Profile Direction)

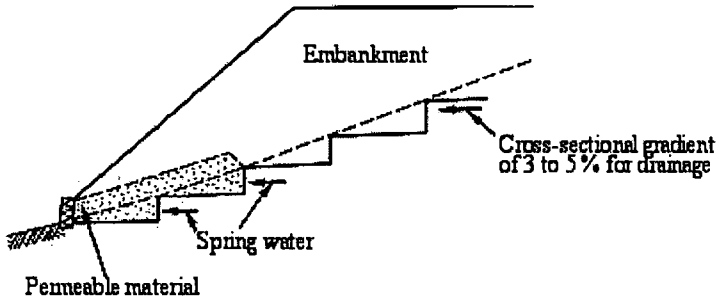


Fig. 4-3-6 Drainage on Ground

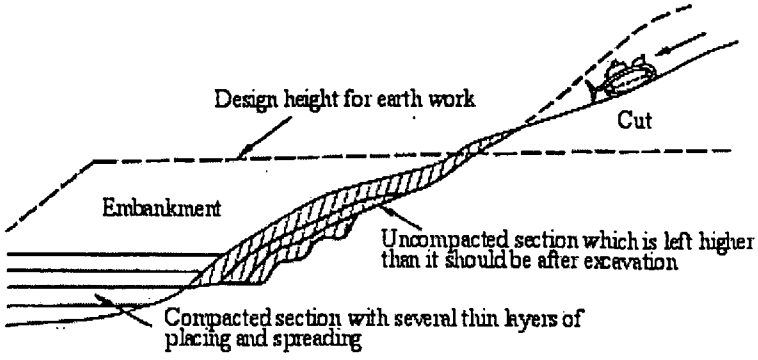


Fig. 4-3-7 Example of Execution often Observed at Boundary between Cutting and Banking Section

(2) Widening of Embankment

Bench cutting is required when widening of an existing road embankment is planned. Careful attention should be paid to preventing excessively large-scale bench cutting as such work could have adverse impacts on the existing embankment. In regard to embankment widening, it is often the case that cracks or faulting occur at the boundary with the existing embankment because of settlement over the years. Although it is difficult to completely prevent such settlement, the degree of settlement can be considerably reduced by the use of high quality soil as the banking material and careful execution, including the proper compaction of each thin layer. If cracks or faulting occur, they should be immediately repaired. What is crucial is to conduct work to prevent the inflow of rainwater into the cracks.

(3) Protection of Trees

In the case of banking work for a mountain road, fencing work to arrest the soil at the road boundary and/or the girdling of tree trucks with a protective material should be conducted so that soil and stones falling towards the bottom of the slope do not damage or bury trees outside the right of way.

(4) Reinforced Earth Method

The reinforced earth method where steel or high polymer materials are horizontally placed inside an embankment has been used in recent years to reinforce embankments and cut slopes. Such steel as reinforcing bars and strips and high polymer materials, including geogrid, are used for reinforcement when the shear strength of the soil is insufficient while high polymer unwoven geotextile is used for the reinforcement of embankments as well as the drainage of water from the soil, separation of the soil layers and prevention of erosion. Moreover, these materials may be laid on the ground surface to maintain trafficability. The employment of these methods taking the soil properties, embankment height, conditions of the original ground and durability of the material into consideration is recommended.

4.4 Countermeasures for Soft Ground

4.4.1 Various Countermeasures for Soft Ground

Commonly used countermeasures for soft ground are counterweight embankment, vertical drain method, sand compaction pile method, slow banking method and their combinations. For the execution of such work, careful consideration is required as listed below.

i) An execution plan capable of ensuring full satisfactory quality, completed work amount and schedule must be formulated while carefully checking the site conditions as well as other conditions specified by the design documents and specifications.

ii) The work must be carefully executed in accordance with the execution plan.

iii) The necessary observation for execution management with special emphasis on stability control and settlement control must be conducted and appropriate judgement must be made based on the observation results. For banking on soft ground, the possible impacts of the weight of construction machinery on the foundation ground should be examined while strictly maintaining the banking speed at which the foundation ground will not fail due to the weight of the embankment. The minimum safety factor for the embankment of 1.2 or higher should be determined. However, when slow, gradual banking is conducted while ensuring the increased strength of the ground or sufficient stability control by field observation, a safety factor of around 1.10 can be used as the design yardstick at the onset of banking work.

iv) If any change of the work situation occurs as a result of execution management, the execution plan should be promptly changed or revised in response to the changed situation.

In the case of methods designed to increase the ground stability, it is essential to execute the work by paying special attention to the work speed to ensure the stabilisation of the ground. In the case of methods designed to facilitate settlement, banking should be conducted early to provide a lengthy period before the open to traffic so that the residual settlement after opening can be minimised. The important points of the various countermeasures for soft ground are listed in Table 4-4-1.

Table 4-4-1 Applicable Soil Types for Each Countermeasure and Important Points for Execution

Countermeasure		Ground Type	Important Points for Execution
Surface Soil Stabilisation Methods	Basal Reinforcement Method	Peaty or Clayey Ground	(1) A suitable reinforcing material should be selected, taking the strength of the surface ground, weight of construction machinery and scale as well as loading width of the embankment load into consideration. (2) A special reinforcement method must be applied to the travelling route of construction machinery with a large ground pressure as localised large tensile strength is generated. (3) The spreading depth of the first layer should not be too thick and material with a good permeability, such as river sand, should be used. (4) When the banking material contains gravel, etc., special attention must be paid to not damaging the installation material.
	Shallow Soil Stabilisation Method	Peaty Ground	(1) Admixing material should be stored at a site which is higher than the ground surface to prevent the invasion of rainwater. (2) When unslaked lime is used, it is necessary to accurately judge the completion time of digestion. (3) When powdered unslaked lime or cement is sprayed and admixtured, special attention must be paid to smoking. (4) The admixing amount of the stabilisation agent is decided based on the properties of the target soil, execution method and results of trial admixture. It is necessary to conduct an admixture test in advance because a sufficient improvement effect may not be expected in the case of ground with a high organic content or sludge (5) As trial admixture does not necessarily represent the site conditions, a safety factor should be added to the in-situ admixture.
	Surface Drainage Method	All Types of Ground - Peaty - Clayey - Sandy	(1) Natural drainage should be arranged using sloping topography. (2) A change of the gradient following the settlement of the embankment should be considered. (3) The infiltration of surface water and seepage water below the embankment from the surrounding cut slopes should be prevented. (4) Drainage ditches should be introduced at narrow intervals to increase the drainage capacity and should be arranged so that the breakage of some ditches does not affect the entire drainage system. (5) The dimensions of a drainage ditch should generally be 0.5 m in width and 0.5 – 1.0 m in depth. (6) It is desirable to conduct the backfilling of drainage ditches with good quality sand or sandy gravel to make them subsurface drainage ditches. (7) When perforated pipes are used as drainage ditches, they should be protected by a filtering material.
	Sand Mat Method	Peaty or Clayey Ground	(1) Finishing stakes should be used with the sand mat method. (2) Spreading is usually conducted with a combination of a dump truck and a bulldozer. (3) Spreading should be evenly conducted to avoid the occurrence of an excessive localised load. (4) The spreading of a sand mat should firstly be conducted in the longitudinal direction at the side sections of an embankment, followed by work from the sides to the centre in the cross-sectional direction of the embankment. (5) When the banking material is soil with poor permeability, special attention should be paid to not disrupting the drainage function at the end sections of the sand mat. (6) The concurrent use of subsurface drainage ditches and/or drainage pipes should be considered to enhance the drainage performance of the sand mat.
Replacement Methods	Replacement Method by Excavation	Peaty or Clayey Ground	(1) It is desirable to select sand, gravel or other coarse soil as a high quality replacement material because of the good drainability and sufficient bearing capacity below the groundwater level. (2) The gradient for an excavated slope surface should be determined within the scope of vertical to a 20% slope depending on the site conditions. To prevent failure of the excavated slope surface, replacement material must be brought in as soon as possible after the completion of excavation.

	Forced Replacement Method with Dead Weight of Embankment	Peaty or Clayey Ground	(1) This method is designed to push out the soft layer with the dead weight of the embankment. However, it is necessary to excavate and remove the ground at the sides of the embankment. (2) Banking starts at the centre of the road and then spreads outwards. Even spreading is essential as soft soil left under the embankment in patches could produce differential settlement after execution.
Counterweight Embankment Method		Peaty or Clayey Ground	(1) It is necessary to avoid high spreading of the embankment and also to compact without fail while grading the banking material for each tin layer in a level manner. The required cross-sectional gradient must be secured for drainage from the embankment surface. (2) The banking speed for a counterweight embankment should not lag behind the work for the main body of the embankment. (3) When it is confirmed that the ground strength under the counterweight embankment has reached the required value during or after execution, it is possible to remove the counterweight embankment to a pre-determined height to transfer the removed soil to the main body of the embankment.
Reinforced Embankment Method		Peaty or Clayey Ground	(1) The reinforcing material must have a high tensile strength and should preferably be capable of following the deformation of the ground. (2) Special attention must be paid to compaction of the banking material around the reinforcing material to enhance the unification with the embankment and the fixing effect by the dead weight of the embankment.
Lightweight Embankment Method		All Types of Ground	(1) Special attention must be paid to material with a low specific gravity as it may be subject to strong buoyancy once inundated. (2) Material which can degrade with ultraviolet rays or petroleum oil, etc. must be covered by top soil, etc. (3) Block-shaped lightweight material must be securely connected.
Slow Banking Method		Peaty or Clayey Ground	(1) While the height of the first stage embankment is determined by the initial strength of the ground, special attention is required as the initial strength may decline depending on the type of ground. (2) A settlement gauge, pore water pressure gauge and other measuring apparatus should be used to measure the settlement speed and changes of the pore water pressure, etc. so that the stability of the embankment and the stage of progress of consolidation can be checked to control the banking speed.
Surcharge Methods	Surcharge Method by Banking (Pre-Loading Method)	Peaty or Clayey Ground	(1) As the correlation between settlement and time is generally difficult to predict, sufficient field observation must be conducted during execution to monitor any failure of the ground. Based on the observation results, the residual settlement after removal of the load and the timing for removal of the load should be decided. (2) If the prevention of settlement is the sole purpose, the application of as large a surcharge as possible for a long period of time is highly effective. (3) In the case where stability of the ground is difficult to secure or the loading time is quite long, consideration must be given to the simultaneous use of the slow banking method or the vertical drain method. (4) As removal of the pre-load leads to elastic upheaval (relief), it is necessary to raise the height by this upheaval. Special attention must be paid to peaty ground because of the possibility of large upheaval deformation.
	Vacuum Consolidation Method	Peaty or Clayey Ground	(1) Efforts must be made to maintain constant negative pressure by paying careful attention to execution control, including maintenance of the airtight membrane and viscosity cap. (2) Theoretically, the surcharge is approximately 10 tf/m^2 but only 5 – 6 tf/m^2 can be usually expected because of the pressure loss due to air leakage, etc. When a larger load than this is required, it is necessary to simultaneously employ the embankment loading method, etc. (3) When used together with the vertical drain method, a positive effect may not be expected to take place due to the pumping up of a huge quantity of water from the sand layer in the soft layer. (4) Because of the high operation cost of a vacuum pump, this method is unsuitable for loading for a long time. The execution depth is some 25 – 35 m although it depends on the specific type of construction machine.

	Groundwater Lowering Method	Clayey Ground	(1) This method is effective when the coefficient of permeability of the sand layer above the soft layer is in the range of $k = 10^{-1} - 10^{-4}$ cm/sec. If a well point is used, pumping up operation is generally feasible up to 5.5 – 6 m from the head loss. (2) When the source for groundwater replenishment is located nearby, the pumped up water quantity becomes extremely large. (3) As lowering of the groundwater level could have an adverse impact on the surrounding area, it may be necessary to encircle the execution section with a water barrier wall.
Vertical Drain Methods	Sand Drain Method	Peaty or Clayey Ground	(1) A sand mat is introduced on the ground surface prior to the execution of sand drain. (2) While there are several types of sand drain, including the vibro-hammer type, auger type and sandbag type, the general execution depth is up to some 25 m below the ground surface. (3) Sand drains must be placed in exact positions based on accurate surveying so that their interval is exactly as set forth in the design documents. (4) The mandrel guide must always be kept vertical. If the design demands the insertion of sand drains in the foundation of a soft layer such as a sand layer or gravel layer, the sand drains must penetrate the foundation layer without fail. (5) While a fairly fast driving speed is permitted, the speed of pulling out the mandrel must have a sufficient margin in connection with the sand filling and pressurised air speeds to ensure the execution of continuous drain. (6) In regard to sand filling, it is essential to check the sand volume. A relatively small sand input volume is preferable. (7) In the case of a vibro-hammer type or driving type sand drain, the pressurised insertion of a mandrel into the ground could have a harmful impact on nearby structures due to deformation of the ground or vibration during execution. It is, therefore, necessary to employ a countermeasure, such as the introduction of an isolation trench.
	Cardboard Drain Method	Peaty or Clayey Ground	(1) The execution depth of the cardboard drain method is approximately 20 m. (2) A sand mat should be created on the ground surface prior to the placing of cardboard drains. (3) Compared to sand drains, cardboard drains have a narrower interval, making careful control of the placement interval necessary. (4) Strict control is required to ensure the continuity of the cardboard. Constant attention must be paid to the amount of cardboard in the cardboard drums to confirm the placement of cardboard in the required positions. If the execution result is unsatisfactory, such as lifting of the cardboard, fresh cardboard must be inserted. (5) The outcrop section at the top of the cardboard must be swiftly protected.
	Sand Compaction Pile Method	All Types of Ground	(1) An execution depth of some 25 – 35 m is possible depending on the machine used. (2) The placement positions and sand pile depth must be strictly controlled. (3) Whether or not the placed sand piles satisfy the required diameter must be checked based on the relationship between the sand input volume and the achieved depth. (4) In the case of cohesive soil ground, the strength of cohesive soil could substantially decline due to agitation causing by piling work. It is, therefore, essential to confirm the improvement effect by means of an in-situ test. (5) When sand piles are inserted through an impervious layer to penetrate a pressurised aquifer with a large head, boiling may occur along the sand piles and careful attention is required to prevent this from occurring. (6) Careful attention must also be paid to vibration and noise during execution. If necessary, an isolation trench or other countermeasures should be employed.

Vibro-Compaction Methods	Vibro-Floatation Method	Sandy Ground	(1) When a thin layer of silt, etc. is present in the ground, the piles could be bent and could disrupt the fall of replenishing material. The employment of a countermeasure, such as the attachment of a ring to the rod or the drilling of a large hole in the silt layer, is necessary. The execution depth is approximately 8 m. (2) As compaction is difficult near the ground surface, rolling compaction is necessary after execution. (3) Gravel, coarse sand or slag are often used as the replenishing material. It is said that the effect is diminished when the maximum grain diameter exceeds 5 cm. (4) Careful attention must be paid to vibration during compaction if there is a structure nearby.
	Rod Compaction Method	Sandy Ground	(1) The execution point interval is 1.3 – 2.5 m while the improvement depth is approximately up to 25 m. (2) The improvement effect is reduced in the case of ground with a high content of fine grain soil of not more than 74 μm in diameter. Careful attention is required when the application of this method is planned for ground with a fine grain soil content of 15% or higher. (3) Careful attention must be paid to vibration, noise and lateral displacement during execution. If necessary, a countermeasure(s) should be employed.
	Falling Weight Compaction Method	Sandy Ground	(1) Because of few applications of this method in the past, trial execution must be conducted to determine the execution specifications together with a confirmation survey during execution. The execution depth is approximately 10 – 15 m. (2) The improvement effect is said to decline with ground with a high fine grain soil content of not more than 74 μm in diameter and, therefore, special attention must be paid to establishing a suitable drop height as well as interval between the percussion points. (3) The structures around the work site may be deformed due to vibration caused by percussion and the necessary countermeasure(s) should be employed to prevent this. (4) A sand mat should be introduced to make the groundwater level 1.5 – 2.0 m below the execution plane for the purpose of preventing standing water in the percussion holes.
Soil Solidification Methods	Deep Mixing Method of Soil Stabilisation	Peaty or Clayey Ground	(1) The execution depth is approximately up to 30 m. (2) Full execution control on the stabilisation material supply volume and pull-out speed, etc. must be performed to ensure uniform improvement. (3) When the improvement of soil with a high organic content, such as peaty ground, is intended, a large volume of stabilisation material is generally required to achieve the required strength. (4) A laboratory admixture test can provide a yardstick for the required stabilisation material volume. However, improved strength is often affected by the state of ground stratification and execution accuracy, etc. It is, therefore, essential to confirm the improvement effect by an in-situ test. The volume of stabilisation material to be added may require reconsideration. (5) Careful attention should be paid to the possibility of the adverse impacts of a volumetric change, air pressure or injection pressure associated with the supply of the stabilisation material on nearby structures.
	Lime Pile Method	Clayey Ground	(1) The execution depth is approximately up to 30 m. (2) The pipe diameter is mainly 0.3 – 0.5 m. (3) The pile interval is mainly 0.75 – 1.5 m. In the case of the driving type, lateral displacement of the ground may occur depending on the pile interval, requiring careful attention. (4) Some 1 m at the head of the lime piles should be made a void for backfilling with clay or mountain sand. (5) As unslaked lime generates heat when absorbing water, it is essential to wear proper clothing and gloves to avoid being burned. (6) When the air pressure adjustment is insufficient after pulling out the casing following the injection of unslaked lime or when the casing is pushed up by unslaked lime part of the said lime may blow up. It is, therefore, essential to wear protective clothing and goggles during the work to pull out the casing.

	Chemical Grouting Method	Sandy or Clayey Ground	(1) When a structure exists near the injection hole, deformation may occur due to the injection pressure or swelling of the ground due to injection. It is, therefore, necessary to careful execute the work while paying proper attention to the injection pressure and injection volume. (2) When unconsolidated chemical outflows beyond the intended area of injection, the spillage can cause adverse impacts on the surrounding environment, including the contamination of groundwater and death of vegetation. The proper selection of the injection method as well as the chemical and the proper storage of the chemical are necessary. In addition, waste water from the washing of surplus material and machinery must be disposed of in accordance with the waste water treatment standards set forth by the relevant law and/or regulation. (3) The quality of groundwater and water in public areas must be monitored and checked at appropriate times before, during and after the work. (4) The locations and types of underground structures must be surveyed in advance to avoid any damage by the injection hole drilling work.
	Freezing Method	Clayey Ground	(1) Swelling of the ground due to freezing or settlement due to defreezing of the ground may occur. A silt or clay layer containing fine grains of 20 μm or less are particularly prone to these phenomena. (2) Freezing is often prevented in the case of ground where a groundwater flow is present. The threshold flow rate is said to be approximately 2 m/day for the brine method and approximately 10 m/day for the cold liquefied gas method.
Structural Method	Pile Works	Peaty or Clayey Ground	(1) Careful attention should be paid to the method where a concrete slab is placed above the pile head as settlement may create cavities below the slab. (2) Careful attention should be paid to the method where a concrete cap is placed above the pipe head as the differential settlement of the piles may have an adverse impact on the road surface. Careful attention is also required in regard to possible damage to the concrete caps at the time of the rolling compaction of the embankment. (3) In regard to the method combining piles with reinforcing bars or a geotextile, the reinforcing bars or geotextile must be firmly connected to the pile heads. (4) When houses or structures exist near the work site, careful attention must be paid to the vibration as well as lateral movement of the ground caused by piling. (5) As differential settlement may occur at the boundary between an improved section by pile works and an unimproved section, careful attention must be paid to grading at the improved section.
	Sheeting Works	All Types of Ground	(1) Careful attention must be paid to an extremely hard ground base as the sheet piles could bend during driving work. (2) When relatively large settlement occurs during banking work, a countermeasure(s) to prevent breakage of the tie rods is necessary. This countermeasure may be the introduction of slits to the sheet piles so that vertical movement is allowed at the connecting point between the tie road and sheet pile or the use of a ring joint for the connection of the tie rods.
	Culvert Works; Viaduct Works		In general, the pre-loading method is often used to stabilise the foundation ground for a culvert. In this case, it is essential to consider the work period and to formulate a plan for the converted use of the surcharge soil in advance.

4.4.2 Settlement and Stability Control of Embankment on Soft Ground

Problematic issues for banking on soft ground are the settlement and stability of the foundation ground which is subject to the embankment load. Both problems sometimes occur simultaneously while either of them can become a problem on other occasions. It is essential for construction work to be accompanied by the field observation described next to deal with these problems.

(1) Field Observation

The main instruments to be installed for field observation are listed in Table 4-4-2. Among these, a surface settlement gauge, surface displacement detection stakes and a surface inver wire extensometer are often used for general banking work. An earth pressure gauge and pipe strain gauge type inclinometer are also used in addition to the instruments listed in Table 4-4-2.

Table 4-4-2 Instruments for Field Observation

Type of Instrument	Purpose of Use		Remarks
	Settlement Control	Stability Control	
Surface settlement gauge	◎	◎	◎: used frequently
Deep settlement gauge	O	O	
Surface displacement stake	-	◎	
Inver wire extensometer	-	◎	
Subsurface displacement gauge	-	O	
Pore water pressure gauge	O	O	

An example of the deployment of various instruments for ordinary banking work is shown in Fig. 4-4-1. Another example of the deployment of field observation instruments at an important or special site is shown in Fig. 4-4-2.

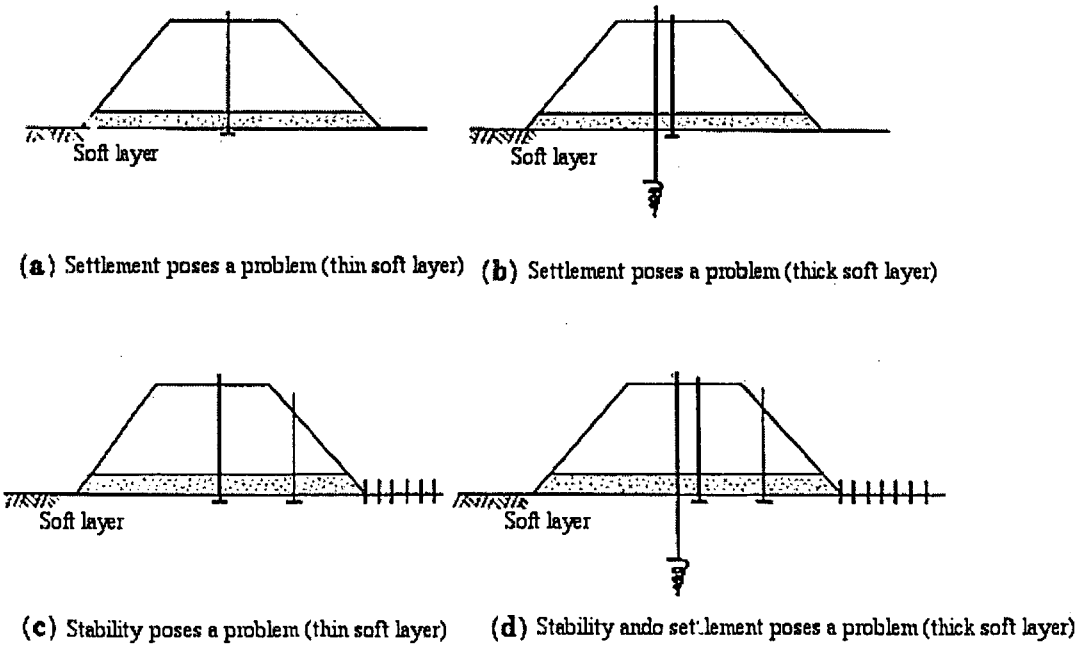


Fig. 4-4-1 Example of Deployment of Field Observation Instruments

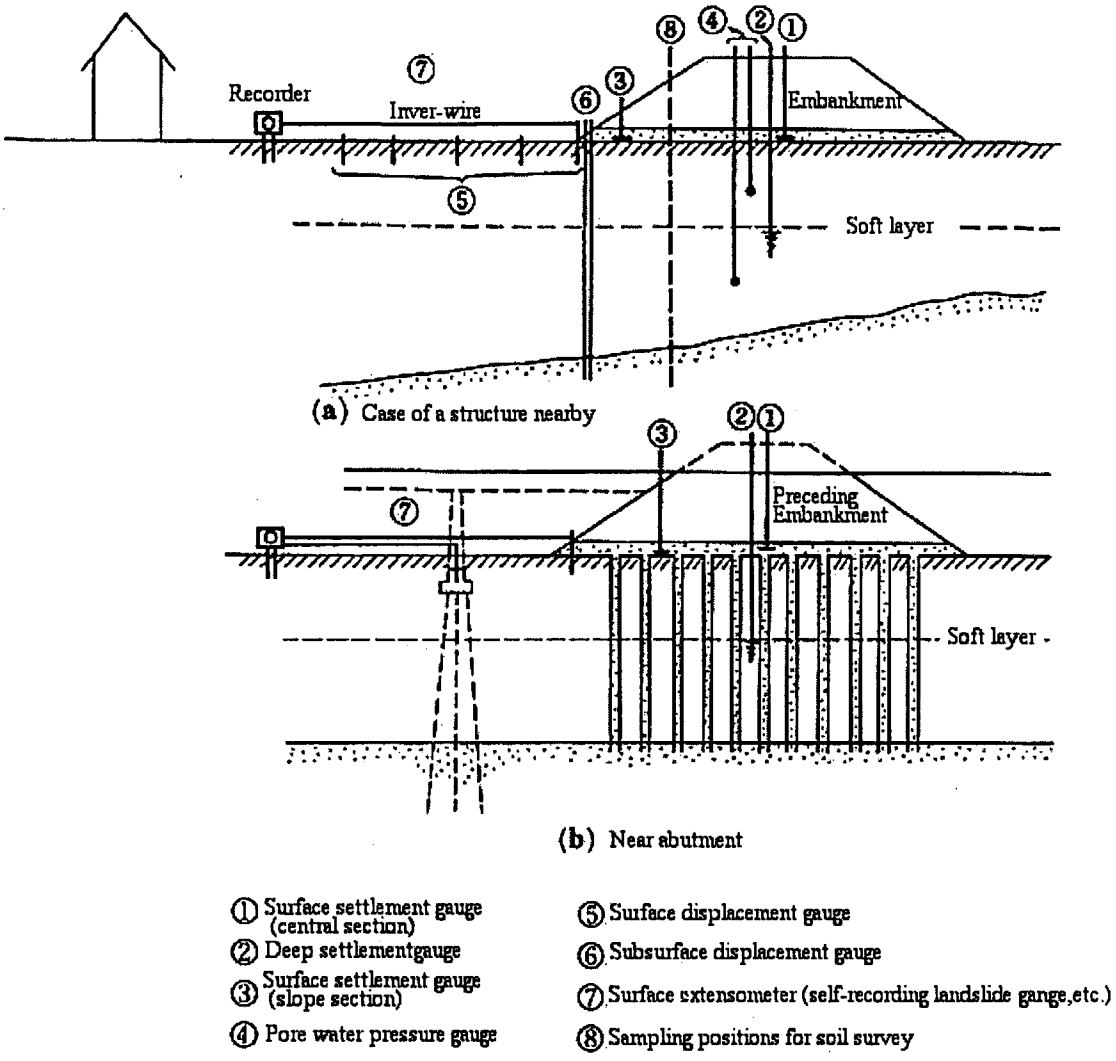


Fig. 4-4-2 Example of Deployment of Instruments at Special Site

Although the measuring frequency and period vary depending on the degree of softness and extent of the soft ground, the importance of the work in question and the schedule, it is essential to establish a management system which is capable of changing the field observation regime to increase the measuring frequency when deformation of the foundation ground is detected or is believed to have occurred during the management period or, conversely, to reduce the measuring frequency when the embankment is found to be stable.

(2) Settlement Control

A structure constructed on soft ground may be damaged by long-lasting major settlement or larger than expected differential settlement. Careful attention is required in regard to settlement control to prevent such damage or to remove the causes of damage to a structure.

There are several methods to estimate the settlement after execution based on the measuring results of settlement in time. Among these, the *hyperbolic method* is applied for short-term estimation during banking work while the *log t method* is used to estimate long-term settlement after the completion of an embankment. Both are believed to be fairly accurate based on their past application results.

The hyperbolic method assumes a decline of the average settling speed along the hyperbolic time-settlement curve and estimates the settlement S_t at an arbitrary time after banking using the following expression.

$$S_t = S_0 + t/(\alpha + \beta \cdot t) \quad \cdots\cdots (4.4.1)$$

Where,

S_t : settlement at the time t

S_0 : initial settlement ($t = 0$)

t : time passed from the reference date

α, β : constants based on actual measurement and can be determined by the correlation shown in Fig. 4-4-3

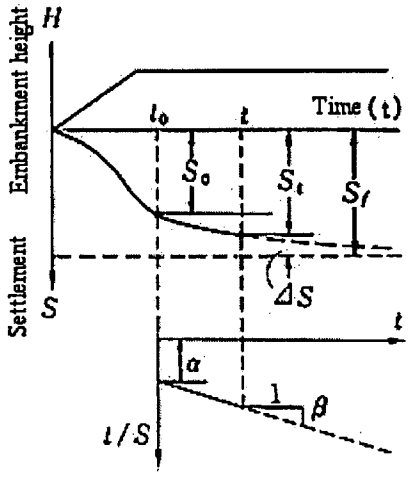


Fig. 4-4-3 Hyperbolic Method

When $t = \infty$, the settlement reaches the final value S_f shown by the following expression.

$$S_f = S_0 + 1/\beta \quad \cdots\cdots (4.4.2)$$

Accordingly, the residual total settlement ΔS at the time of t from the reference date is shown by the following expression.

$$\Delta S = S_f - S_t \quad \cdots\cdots (4.4.3)$$

Soft ground often consists of several soft layers with a different speed of consolidation rather than one simple layer. It is, therefore, desirable to measure the settlement of each soil layer with different consolidation characteristics using a deep settlement gauge, to apply the hyperbolic method to determine the time-settlement curve for each layer and to produce a composite curve to determine the overall settlement.

Meanwhile, the log t method predicts the future settlement using a graph where the settlement is expressed by the arithmetic scale and the time is expressed by the logarithmic scale.

(3) Stability Control

When the construction of an embankment on soft ground is planned, it must be recognised that there is a possibility of the bearing capacity of the foundation ground being insufficient, resulting in considerable deformation or failure.

When deformation of the foundation ground significantly increases, the area of deformation gradually extends with the strength of the ground simultaneously declining. It is, therefore, essential for the field observation results to be quickly analysed so that the work can progress with a constant understanding of the behaviour of the soft ground.

The main principles of stability control are listed below.

① To control the execution of banking at the planned speed.

② To continually measure the degree of deformation and changes in time of the foundation ground as well as the embankment and to analyse the measuring results.

③ To investigate changes of the strength of each soft soil layer by conducting a soil test on undisturbed samples collected from the soft ground or an in-situ test, such as the Dutch double-tube cone penetration test, whenever such investigation is necessary.

1) Settlement Measuring Results

As shown in Fig. 4-4-4-(a), settlement near the central part of an embankment shows a tendency to converge to a certain value if the soft ground is stable. When the foundation ground approaches the point of failure, the amount of settlement in time is changed to the pattern shown in Fig. 4-4-4-(b) with a rapid increase of the settlement speed.

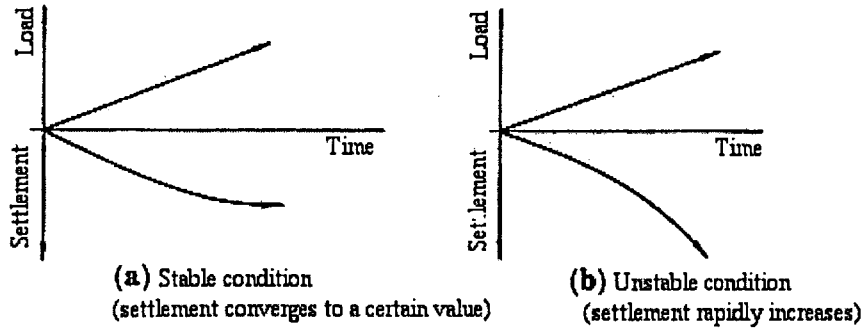


Fig. 4-4-4 Change of Settlement in Time and Stability of Foundation Ground
(Near Central Section of Embankment)

2) Measuring Results of Side Ground of Embankment by Surface Displacement Detection Stakes and Surface Extensometer

Lateral displacement in time is hardly observed or slight displacement towards the banking side is observed when the foundation ground is stable as shown in Fig. 4-4-5-(a). When deformation of the foundation ground becomes considerable, lateral displacement rapidly increases towards outside the banking as shown in Fig. 4-4-5-(b).

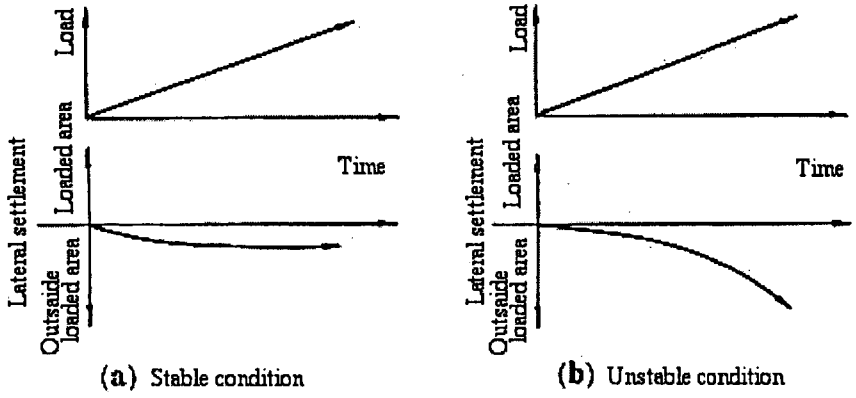


Fig. 4-4-5 Change of Lateral Displacement in Time

Vertical displacement in time is hardly observed or a tendency towards settlement is observed when the foundation ground is stable as shown in Fig. 4-4-6-(a). When the foundation ground becomes unstable with significant deformation, displacement shows a tendency towards upheaval as shown in Fig. 4-4-6-(b). This displacement continues even if banking is suspended.

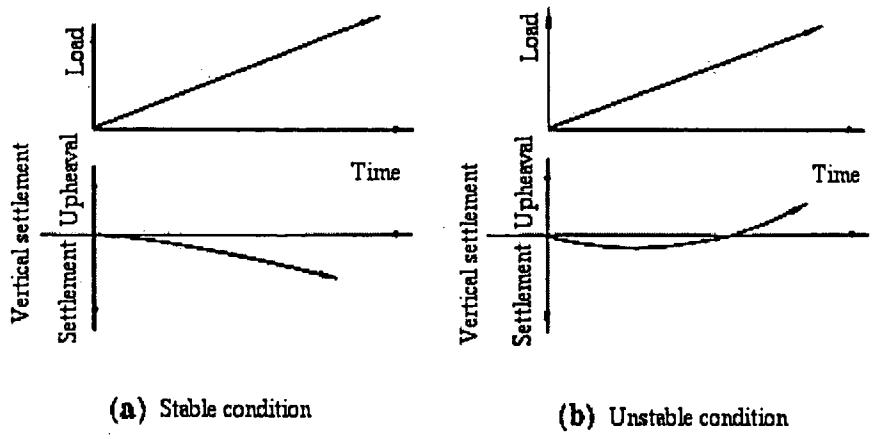


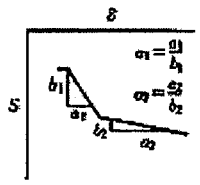
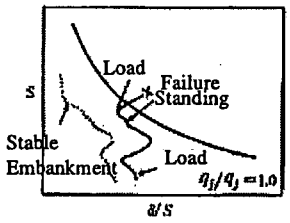
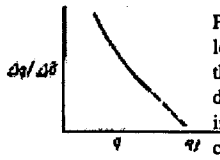
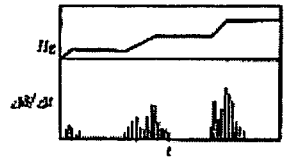
Fig. 4-4-6 Change of Vertical Displacement in Time

3) Stability Control Method Using Quantitative Index

A number of methods to quantitatively conduct the stability control of embankments based on field observation results have been proposed and the main methods are listed in Table 4-4-3.

The examples of the standard control values shown in the table are empirical values for embankments for expressways and, therefore, can be used as practical yardsticks. However, as these values change depending on the banking as well as ground conditions, it is desirable to finalise these values with reference to the relevant values for existing similar ground and the results of the test construction and preceding embankments.

Table 4-3-4 Quantitative Stability Control Methods

Control Method	Description	Example of Reference Value
S－δ Method	 S and δ are plotted. Predict a failure as $a_2/a_1 - a_1$ exceeds a certain value.	Balance between compressive deformation and shear deformation can be easily established to detect the early signs of failure Typical reference values for control: $a_2 \geqq 0.7$ $a_2 \geqq a_1+0.5$
S-δ/S Method	 S and δ/S are plotted. Predict a failure by checking if their track approached the reference failure line or not.	Effective method to understand ground behavior throughout banking operation. Typical reference value for control: $S > 5.39 \exp\{1.28(\delta/S)^2 - 3.41(\delta/S)\}$
Δq/Δδ－q Method	 Predict the breaking load q_f by extrapolating the relation Δq/Δδ-q during a gradual load increase (shown by the curve from the bottom right).	Effective method to infer the critical embankment height using data up to a certain banking height. Typical reference value for control: $\Delta q/\Delta\delta \leqq 15 tf/m^2$
Δδ/Δt－t Method	 Predict a failure as displacement of δ per day (Δδ/Δt) exceeds a certain value	It can easily check the displacement per day. Typical reference value for control: $\Delta\delta/\Delta t \geqq 1-2$ (cm/day)

Note; S: settlement at the central part of an embankment, q_j: embankment load at an arbitrary time, q: embankment load, δ: lateral displacement at the slope bottom of an embankment q_f: embankment load at the time of failure, H_E: embankment height

4.5 Slope Work

4.5.1 Cut Slope Work

The normal procedure for cut slope work is to start with rough excavation at the cutting section, followed by shaping and finishing of the slope surface and the execution of slope protection work. Special attention should be paid to the following points for the execution of cut slope work.

i) As it is difficult to fully understand the ground conditions of the cutting section through a preliminary survey, it is essential to regularly check the ground conditions during excavation of the cutting section. Comprehensive judgement should be made on the slope gradient and cutting height by verifying the state of the ground with the shape of the cut slope indicated in the design drawing. If it is recognised that a change is required, the necessary measure should be promptly implemented.

ii) A sufficient number of finishing stakes should be placed for excavation at the cutting section so that the planned excavation line is not exceeded.

iii) In the case of cut slopes where retaining walls were originally planned, the gradient of the cutting slope frequently becomes steeper than the standard values, and thus slope failures often occur during or upon completion of excavation and prior to the construction of retaining wall. Safety management should be strictly carried out particularly during excavation and during successive retaining wall work.

iv) When spring water or seepage water is discovered on the face of the slope, countermeasures, such as weep holes and subsurface drainage ditches, should be implemented.

v) The construction of drainage ditches to be introduced at the top of the slope and berms should be carefully executed so that the face of the slope is not scoured by the splashing of running water and/or overflow.

vi) After executing the slope work, pumice stones likely to fall in the future should be carefully removed with a pick hammer or bar. If it is difficult to remove large pumice stones completely, they should be anchored to the ground by anchor works. The rock fall prevention net is often implemented where the height of the slope is large and where there is the potential danger of minor rock falls during work.

vii) Minor faults found during work should be fully examined particularly for their sizes, directions, degrees of fracture, presence of spring water and so forth, in order to investigate whether they could induce large-scale failures. Even though they seem not to trigger any large-scale failure, small local failures tend to occur frequently, so that these small faults should be properly treated, if necessary, by means of concrete block pitching or weep holes.

4.5.2 Embankment Slope Work

The execution methods for embankment slopes include the method whereby a slope section (width: 0.5 – 3 m) is separately constructed prior to or after banking of main body and the method whereby excess banking on the planned slope section is done first for subsequent cutting and shaping. The suitable method should be decided based on the site conditions. Special attention should be paid to the following pointes for the execution of embankment slope work.

i) The slope section must be fully compacted to prevent surface erosion by rainwater, etc.

ii) Finishing stakes should be introduced to avoid an insufficient embankment cross-section due to the compressive settlement of the embankment and other reasons. These stakes should be removed as soon as the work has been completed to prevent the occurrence of scouring by rainwater.

iii) Slope protection work should be promptly executed.

iv) The concentrated downward flow of rainwater from the top of the embankment to the face of the slope should be prevented.

v) The construction of drainage ditches to be introduced at the face of the slope and berms should be carefully executed so that the face of the slope is not scoured by the splashing of running water and/or inundation.

(1) Compaction of Embankment Slopes

An embankment slope must be fully compacted by machinery. Once rainwater infiltrates the face of a slope, there is a risk of slope failure, mainly due to lack of uniformity near the face of the slope. Full compaction by rolling over each of the lateral thin layers is generally considered to be effective to prevent such lack of uniformity. Compaction of the face of the slope can be directly conducted by a compacting machine when the slope gradient is gentle (approximately 1:1.8). Another method in use is excess banking on the slope for subsequent cutting and shaping to achieve the required slope.

Slopes to be made with materials such as clay or volcanic cohesive soil with high water contents, which cannot be completely compacted, should be very carefully worked out by paying special attention to the stability of the whole slope. Any deformation of finishing stakes or swelling of the slope should be carefully observed during work. If any indications are found, their causes and future stability should be examined.

Earth should be replaced or slope gabions or the drainage work such as horizontal drainage layer in embankment should be provided as needed.

Where the main body of embankment is to be made with the coarse-grained soils such as gravel or sandy soil and therefore any problem is anticipated such as erosion or difficulties in vegetation, slope is often covered with top soil. In this case, the boundary between top soil and already executed fill body should be formed by properly mixing them together and be compacted without leaving a clear boundary (refer to Fig. 4-5-1).

If possible, it is desirable to have a gentle slope ratio of about 1:1.8 for the embankment slope and to design a thickness of about 2 to 3m for blanket soil in order to make machine execution possible (refer to Fig. 4-5-2).

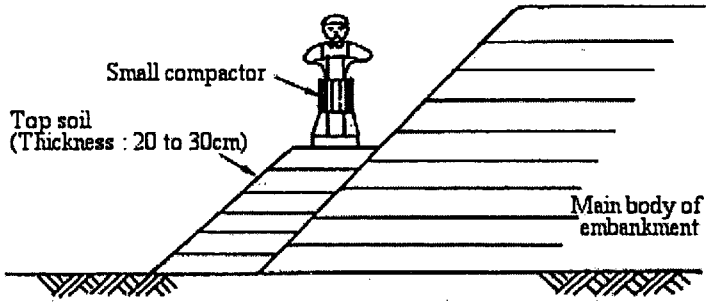


Fig. 4-5-1 Compaction of Top Slope

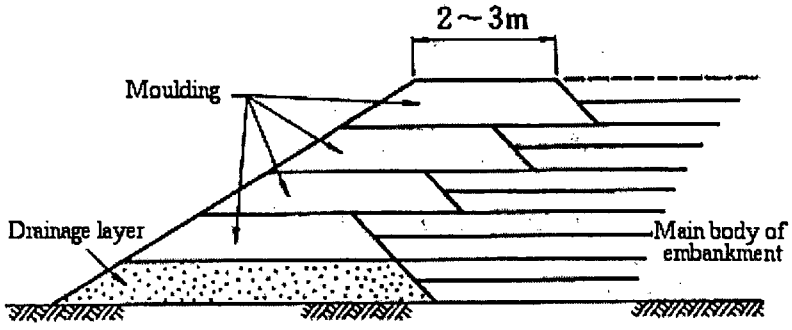


Fig. 4-5-2 Example of Blanket Soil considering Machine Execution

(2) Slope protection during Work

Temporarily finishing slopes are most unstable until the protection work is performed, and will be easily eroded by rain water or others. Therefore, the slope protection by vegetation or longitudinal drainage facilities should be provided as quickly as possible. However, as temporary measures until the slope will be fully protected, it is recommended to employ filling work shown in Fig. 4-5-3(a) in order to avoid the concentration of surface water on the surface of slope. Also, if the executed embankment is left for a long

time after completion until paving work, it is desired to install a temporary ditch using a soil cement mixture as shown in Fig. 4-5-3(b), if required, in order to prevent the occurrence of any problem due to concentrated flow of rain water. In the case of high embankment with erosible soil, temporary drain ditches as shown in Fig. 4-5-4 is effective in order to prevent surface water from flowing down the slope.

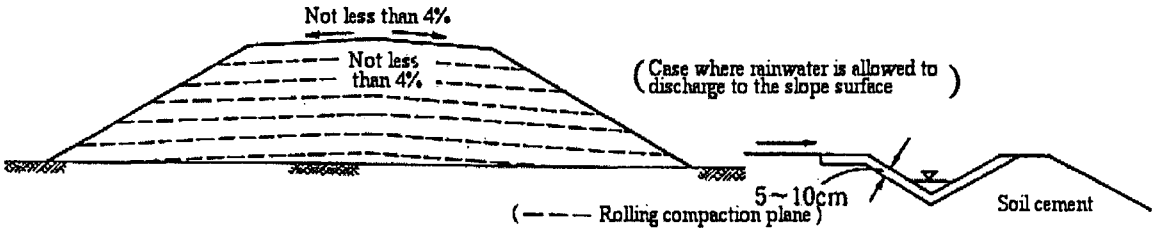


Fig. 4-5-3 Treatment of Surface Water during Banking

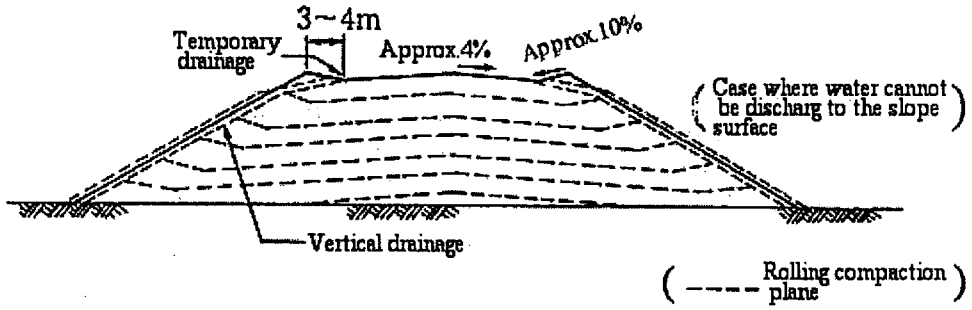


Fig. 4-5-4 Example of Temporary Drainage during Banking with Soil Vulnerable to Erosion

4.5.3 Slope Protection Work

Slope protection works are performed to protect the slopes from erosion or weathering by covering them with vegetation or structures and also to stabilize the slope by means of drainage works or retaining structures.

i) Slope protection work should immediately follow the completion of cutting or banking work. The execution timing for planting should be carefully decided.

ii) Slope protection work, be it planting or structural work, is essentially not capable of withstanding earth pressure. A different type of countermeasure is, therefore, required if there is a risk of facing earth pressure. If failure is anticipated at part of a cut slope, such necessary treatment as the removal of soil from that part should be conducted.

iii) For slope protection for full cover of slope by spraying, attention should be paid to the drainage of spring water or seepage. And spring water from slope should be removed as much as possible for other protection work.

(1) Vegetation

Success or failure of vegetation is governed by the growth of plants. Therefore, the species and execution method should be carefully selected in accordance with the following conditions, and the conditions capable of assuring the complete growth of the selected species should be provided.

① Area, gradient, height of slope
② Condition of adjacent area
③ Soil conditions (physical and chemical composition, water content and hardness of soil, unevenness, presence of spring water, etc.)
④ Weather conditions (air temperature, rainfall, slope direction, degree of sunshine)
⑤ Other (degree of difficulty in securing the local materials (such as earth and water), their qualities, and conditions of access road for bringing in the machinery and materials)

(2) Slope Protection Work using Structure

Special attention should be paid to the following points for the execution of slope protection work using a structure.

i) It is extremely difficult to accurately predict the ground conditions prior to excavation. Therefore, if the ground conditions discovered considerably differ from the assumptions at the design stage, the suitability of the planned slope protection work should be carefully examined. When a retaining wall or cribwork is planned, whether or not the existing ground can be used as its bearing stratum must be judged with a view to implementing appropriate measures if necessary.

ii) In the case of a long slope, it is desirable to execute protection work in line with the progress of excavation without waiting for the completion of excavation. This is because of the fact that the execution of protection work after the completion of excavation is uneconomical due to the need to transport the protection materials from the bottom of the slope and also because of the fact that the face of the slope is left bare for a long period of time, resulting in loosening, weathering and erosion of the ground.

iii) When sealed-type protection work, such as mortar/concrete spraying or pitching, is executed, the implementation of spring water control and drainage measures is required. In regard to cribwork,

sufficient compaction as well as spring water control and drainage measures are necessary to prevent the outflow of the filling materials.

iv) Prior to the execution of mortar/concrete spraying or spray cribwork, unwanted sediment, dust, pumice stones and stumps, etc. should be carefully removed from the face of the slope to prevent the detachment, unevenness or cavities behind the sprayed material. Careful attention must be paid to the timing of execution and the meteorological conditions at the time of execution to prevent (i) poor adhesion to the ground during the dry season when cracks are more likely to occur and (ii) a decline of the quality of the spraying material.

v) As the yield strength and effects of rock bolts and ground anchors are considerably affected by the execution skill, careful attention must be paid to drilling, insertion of steel, fixing and grouting. In addition, drilling should be carefully conducted so as not to loosen the surrounding ground and the tension members should be given the proper treatment to prevent their corrosion.

vi) Proper consideration should be given to safety control during the execution of slope protection work as it may be necessary to conduct such work at a high place or on a steep slope. The likelihood of the presence of unstable pumice stones or the instability of the bare slope during or immediately after execution also demands careful safety control.

4.6 Countermeasures for Rockfall, Landslide and Debris Flow

Slopes or mountain streams requiring the execution of countermeasures for rockfall, landslide and/or debris flow are generally steep and the construction conditions are often poor because of the proximity of housing near the site and other reasons. It is, therefore, essential to pay special attention to safety measures not only for the construction site but also for the adjacent area in the formulation of an execution plan.

During execution, careful attention is particularly required during and immediately after torrential rain because of the high risk of rockfall, slope failure and/or debris flow. If necessary, a raingauge, rockfall detector, landslide monitoring system and/or debris flow monitoring system should be installed to ensure personal safety as well as the safety of nearby facilities during the period of execution.

4.6.1 Countermeasures for Rockfall

Special attention should be paid to the following points for the execution of countermeasures for rockfall.

(1) Rockfall Prevention Work

i) If the site for rockfall prevention work is characterised as a type of topography which is prone to collect rainwater or seepage water or geology which is prone to erosion, the foundation for such work becomes liable to erosion. At such a site, it is necessary to shape not only the target slope but also nearby slopes and to implement groundwater and surface water control measures.

ii) Rocks which are considered to be extremely dangerous should be made to fall or should be removed at the beginning of the execution period. If this is found to be impossible, the state of penetration and cracks of such rocks should be estimated and a simple measure or temporary facility should be implemented in anticipation of their eventual fall.

iii) The foundation for foot protection work, concrete pitching or cast-in-place concrete cribwork, etc. should be executed above ground with a sufficient bearing capacity.

iv) A rock anchor or wire rope fixing must be fixed to a foundation which provides a sufficient bearing capacity to withstand the weight of pumice stones or boulders.

(2) Rockfall Protection Work

Fastening of the members for such rockfall protection work as a rockfall protection net and rockfall protection fence should be conducted several times while ensuring a good balance.

4.6.2 Countermeasures for Landslide

Special attention should be paid to the following points for the execution of countermeasures for landslide.

(1) Control Work

i) When corrugated pipes, etc. are used for channel work, they must firstly be fixed to the ground by wooden stakes or similar to prevent their lifting.

ii) When a large-scale drainage conduit is constructed at the lower part of a slope, careful attention, including work execution at a short section at a time, should be paid to not damaging the stability of the landslide mass.

iii) For the execution of lateral boring, a suitable bit for the geological conditions must be used. In addition, the conformity of the actual geological conditions with the design conditions should be constantly checked. If there is any major discrepancy, a change of the design may be necessary.

iv) When the length of lateral boring becomes quite long, the drilling hole could be bent at a gravelly soil layer or heterogeneous layer. Careful attention is, therefore, required, including confirmation of the frontal position of the boring hole using a bending gauge, during excavation.

v) In regard to earth removal work, the earth should, in principle, be removed downwards so as not to damage the stability of the slope. This type of work should be avoided during the rainy season.

vi) The face of a slope after earth removal generally has a high level of permeability and is prone to softening and even failure due to rainwater. It is, therefore, necessary to implement slope drainage work immediately after earth removal, followed by appropriate slope protection work.

(2) Prevention Work

i) In regard to pile work, when landslide movement is serious enough to be clearly visible, piles driven into the ground only function individually unless all of the planned piles are driven at the same time. As the positive effects of piles cannot be expected to materialise, pile work should be avoided under this condition.

ii) Piles are usually inserted into the ground to avoid harmful vibration to the ground. When a large quantity of drilling mud is used for the drilling of holes, loosening of the ground and/or the creation of water paths may occur with adverse impacts regarding landslide. Careful attention should be paid to reducing the quantity of drilling mud and to using thicker slurry.

iii) For the execution of shaft work, excavation and muck haulage are conducted in a narrow place same as in the case of drainage well work, making it essential to pay proper attention to rockfall.

iv) For the execution of anchor work, the drilling speed and slime, etc. should be checked during drilling with the design ground conditions. In places where the anchor fixing ground is of poor quality, a borehole lateral loading test should be conducted to check the presence of the design ground strength and properties.

v) When the compressive strength of the grout for the anchor body exceeds the required value, either a tensile or confirmation test should be conducted for all of the anchors to determine whether or not the design anchor strength can be achieved. After the test, tension and fixing work should be conducted with the predetermined tensile force. In this case, tension work should be conducted to achieve a good balance of all of the anchors.

4.6.3 Countermeasures for Debris Flow

Special attention should be paid to the following points for the execution of countermeasures for debris flow.

i) Countermeasures for debris flow are generally executed inside or near a mountain stream. The formulation of an execution plan which involves the construction of a temporary drainage channel as well as a temporary cofferdam to completely manage running water is necessary so that no disaster occurs during the work period. For this reason, the execution of countermeasures for debris flow during the dry season is desirable.

ii) A survey should be conducted on the rainfall conditions at the time of the occurrence of debris flow at the subject as well as nearby mountain streams to establish the hazardous rainfall level for the occurrence of debris flow. When rainfall exceeding this level is predicted to occur during the work period, it is necessary to adopt safety measures, including temporary suspension of the work to evacuate the area, sooner rather than later.

iii) Dumping of the surplus soil from the work into a mountain stream should be avoided as such soil can become a new source of debris flow.

4.7 Construction of Retaining Wall and Culvert, etc.

4.7.1 Construction of Retaining Wall

For the construction of retaining wall, it is necessary to check at the time of excavating the foundation ground whether or not the actual topography and geology conform to the relevant assumptions at the design stage. If the shape and/or geological composition of the slope at the back of the retaining wall or the conditions of the strata of the foundation ground differ from those in the design drawings, it may be necessary to consider a change of the shape, including the height of the retaining wall or even a change of the retaining wall type if necessary. Should any structure exist nearby, execution work must be conducted based on detailed survey results so that such damage as settlement, shifting and/or tilting are not caused to the structure.

(1) Excavation at Back of Retaining Wall and of Foundation Ground

As the excavation gradient at the back of the planned retaining wall and of the foundation ground is often required to be a short-term stable gradient, special attention should be paid to ensuring the safety of the work. The general precautions required for this type of work are listed below.

i) Entire excavation of the ground should be avoided as much as possible and partial excavation (pit excavation) is preferred.

ii) Precautions are required for excavation at the back of the planned retaining wall immediately after rain as the soil there is liable to failure.

iii) Excavation should proceed while preventing failure by sheathing if necessary.

iv) Careful attention should be paid to the conditions of the surrounding ground during excavation. Excavation in high risk places, such as at the bottom of a cliff, should preferably use a backhoe or similar equipment and a watchman should be assigned.

v) Foundation work should be conducted by verifying the required bearing capacity, etc. of the foundation ground.

(2) Foundation Work for Retaining Wall

There are two types of foundation for a retaining wall: spread foundation or pile foundation.

The treatment of the foundation described below is required to fully ensure the resistance of a spread foundation to sliding. In general, an allowable bearing capacity of approximately 300 kN/m^2 is required of the ground to support a retaining wall.

When the foundation ground consists of a rock mass, cutting should be conducted at the bottom position of the base, pumice stones, etc. produced by excavation should be completely removed and the surface of the rock mass should be thoroughly washed for direct construction of the base above it and the base should be constructed above levelling concrete or mortar layer. When the foundation ground consists of soil, cobble stone or crusher run should be laid on the excavated ground and levelling concrete should be cast if necessary above the fully compacted cobble stone or crusher run for construction of the base on top of it. When part of the ground below the base is soft due to titled foundation ground or other reasons, such part may be replaced by concrete.

When spread foundation is applied on the soft ground and thickness of soft ground layer is thin, such soft ground may be excavated to create new foundation ground consisting of good quality replacement material. In this case, it is desirable to conduct a full soil test on the replacement material together with a plate load test prior to execution work of the base to confirm the existence of the required bearing capacity.

(3) Main Body of Retaining Wall

It is desirable for a concrete retaining wall to be made as an integral structure of a base and vertical wall. When lateral construction joints are necessary for a plain concrete structure, such as a gravity retaining wall, keyed joints or grooves should be introduced for such joints. Alternatively, steel bar should be inserted at appropriate intervals. When concrete is cast onto already hardened concrete, loose aggregates, low quality concrete and/or laitance on the surface of the hardened concrete must firstly be completely removed before sufficient water is absorbed. Cement paste or mortar of which the composition is similar to that of the mortar used in the hardened concrete should be applied to the surface of the hardened concrete, immediately followed by the casting of fresh concrete for its close adhesion to the hardened concrete.

V-shaped vertical construction joints should be created on the surface of the retaining wall and their interval should preferably be less than 10 m. Reinforcing bars should not be cut at these joints. The reasons for creating V-shaped vertical construction joints are the prevention of breakage at the corners and the prevention of small cracks on the wall surface.

Expansion joints should generally be introduced at intervals of less than 10 m for a gravity retaining wall and 15 – 20 m for a cantilever retaining wall, etc. and reinforcing bars should be cut at these joints. The introduction of expansion joints as discontinued sections of the foundation ground is necessary to prevent cracks, etc. due to differential settlement.

There are two types of concrete block retaining walls, i.e. dry masonry wall and wet masonry wall. Meanwhile, there are two types of masonry work, i.e. uncoursed masonry and coursed masonry. The standard practice is wet uncoursed masonry.

Weep holes must be executed without fail at the positions indicated in the design drawings. For the execution of backfilling concrete for a concrete block retaining wall, backing forms should preferably be used and backfilling should be frequently conducted with small quantities in line with the rise of the wall so that the wall will not fall down.

4.7.2 Construction of Culvert

Prior to the execution of a culvert construction, it is essential to confirm the conformity of its position and height with the local topography. In the case of a water channel culvert, special attention should be paid to the elevation of the lower end of the culvert.

(1) Excavation of Foundation for Culvert

Although the excavation method differs depending on the planned excavation width and depth and also on the conditions of the ground, the open cut method with sloping sides is generally used. When the excavation depth is deep at a small work site, however, the open cut method with an earth retaining wall is opted for. In either case, special attention must be paid to preventing failure during the work, especially in the case of narrow unsupported excavation because of its liability to failure without proper preparation.

In the case of the open cut method with an earth retaining wall, the state of the foundation ground and its relationship with the groundwater level must be thoroughly surveyed to determine the penetration depth of the sheet piles and others in order to prevent boiling (in the case of sandy ground) or heaving (in the case of cohesive soil ground) during excavation.

When excavation below the groundwater level is planned, the necessary drainage measures must be implemented, taking the state of the ground, suitable excavation method and volume of groundwater into

consideration. In the case of the implementation of a large-scale drainage measure, the possible impacts of lowering of the groundwater level on the surrounding area must be analysed together with due consideration of the suitable disposal of the drained water.

(2) Foundation Work for Culvert

The most important requirement for the foundation for a culvert is to ensure the even distribution of the bearing capacity which is required by the design. For this reason, if the geological or topographical conditions are not uniform, suitable design and execution must be conducted. For example, when a culvert is planned at the boundary between a cut slope and an embankment or at a valley with considerable changes of the geological conditions, the site conditions must be accurately understood at the time of excavation of foundation. If fragile soil which does not meet the design conditions is discovered, replacement with good quality soil is required while ensuring the uniformity of the ground.

If the construction of a culvert above soft ground is unavoidable, it is preferable for preloading to be conducted for the integral settlement of the culvert with the surrounding embankment in addition to the implementation of careful investigation. Pile foundation may be used if the settlement is not acceptable in the case of a water channel culvert and others. In this case, a possible increase of the surcharge and the occurrence of an uneven road surface due to settlement of the surrounding ground should be carefully examined and countermeasures should be introduced if necessary.

In principle, the placing, spreading and compaction of sand, gravel and cobblestones and the casting of a levelling concrete should, in principle, be conducted by means of dry work. However, a fixed concrete foundation for a pile culvert is structurally required unlike a levelling concrete and must be carefully executed to achieve the design shape and quality.

(3) Construction of Main Body of Culvert

The positions of reinforcing bars and forms must be in accordance with the design drawings. The most suitable concrete casting method must be selected in view of the topographical conditions and the casting positions while the supporting work for forms must be compatible with the selected concrete casting method.

Particular attention should be paid to the positions of the construction joints for concrete casting and these positions must be decided based on the design data, taking the stress distribution into consideration. From the viewpoint of structural soundness, casting in the sequence shown in Fig. 4-7-1-(a) is desirable. However,

the sequence shown in Fig. 4-7-1-(b) may be used because of the difficulty of placing forms and/or casting concrete. In this case, the concrete for the corners and the top slab must be cast at least two hours after the casting of the concrete for the side walls. In addition, the cast concrete must be fully cured to prevent the occurrence of settlement cracks.

The installation of a water stop at expansion joints must be executed with special care to prevent the occurrence of voids and/or water leakage. The material to be injected to the joints is an admixture of asphalt and rubber, etc. which must not fluidise at room temperature, must firmly adhere to the concrete and must not cause any adverse impacts on the water stop.

When constructing a culvert with precast concrete or centrifugal reinforced concrete pipes, attention must be paid to ensure its firm settlement on the foundation, to careful execution of the joints and to careful handling of the materials during their transportation so as not to cause any damage.

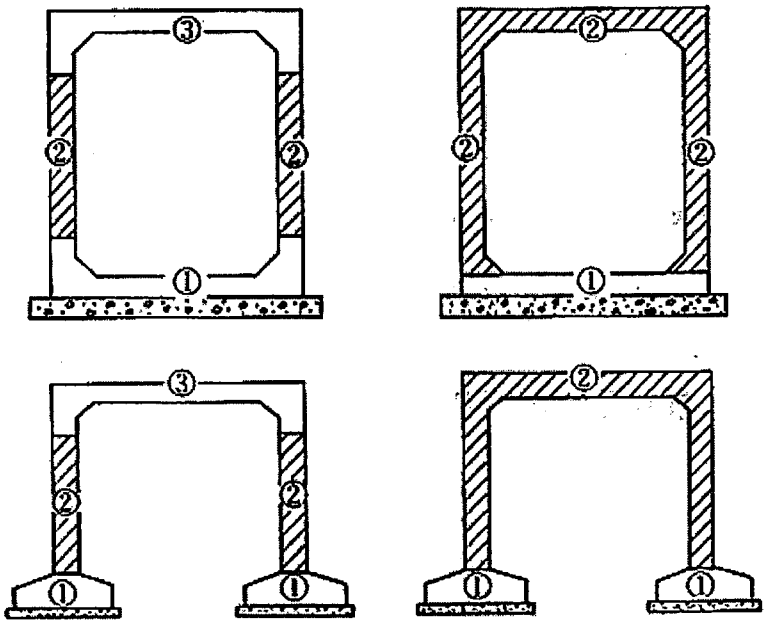


Fig. 4-7-1 Sequence of Concrete Casting

(4) Important Points for Execution of Culvert Construction

In the case of staged construction, it is desirable to execute the culvert and earthwork at the adjoining section of the culvert in accordance with the final cross-section. The work to construct a culvert above soft ground in particular should be executed with the final cross-section from the beginning as differential settlement may occur between the initial work section and subsequent work sections, possibly causing breakage of the culvert. If the adoption of stepped execution is unavoidable in the case of a rigid culvert, sleepers should be

placed in advance. In addition, the reinforcing bars should be installed so as to avoid any problem at the joints in the future and the installation of a water stop(s) is also desirable.

In the case of a flexible culvert, differential settlement by the initial work and differential settlement by additional work may affect each other in a complicated manner and the resulting actual settlement may differ from the predicted pattern of settlement, causing a problem for water flow. Special attention is, therefore, required to ensure proper water flow as planned.

4.7.3 Execution of Backfilling and Approach Cushion

(1) Problems of Backfilling Work

At the connecting section between such a structure as a bridge or culvert and an embankment, the settlement of this section after its opening to traffic may cause faulting on the pavement surface, disrupting vehicle traffic and possibly damaging the structure involved.

Settlement at the connecting section between an embankment and structure may well be the result of the problematic construction of the said section as follows in addition to settlement of the embankment's foundation ground as well as compressive settlement of the backfilling material.

i) When the excavated soil to accommodate the foundation for a structure is soft, such soil may be mixed with backfilling material, causing deterioration of the quality of the backfilling material.

ii) As a backfilling section is often surrounded by an erected abutment or culvert and its wing walls as well as the embankment, the drainage tends to be poor.

iii) As backfilling is the final work to be executed, spreading depth tends to be higher than it should be. Compaction at the backfilling section tends to be insufficient because of the narrow space.

In view of the likelihood of these problems, the following countermeasures are often employed to prevent faulting at the connecting section between a structure and embankment.

① Careful execution of backfilling using good quality material

② Introduction of an approach cushion at the connecting section between a structure and embankment, if necessary

③ Adoption of sufficient drainage measures by means of securing a drainage slope during execution and installing a subsurface drainage ditch(es)

④ Implementation of necessary improvement work, referring to 3.6 – Countermeasures for Soft Ground, to minimise settlement as considerable settlement tends to occur at a connecting section above soft ground

(2) Important Points for Backfilling Work

Good quality material with low compressibility and high permeability should be used as the backfilling material for a structure at a banking section and the work must be carefully executed paying proper attention to drainage.

Table 4-7-1 shows the properties of soil which is suitable as a backfilling material. However, it is often difficult to use such a high quality material in a large quantity. If sufficient compaction can be conducted by a large compaction machine, the use of good quality material is unnecessary. The selection of a material with a good particle size distribution among the different banking materials available is permissible.

Important points for the execution of backfilling work are listed below.

i) The finishing thickness for backfilling and spreading should not be more than 20 cm and compaction should be conducted at a similar intensity to the compaction of the subgrade.

ii) The backfilling material should be evenly spread by a small bulldozer or by hand and overfilling using a dump truck or bulldozer should be avoided.

iii) The machine to be used for compaction should be as large as possible while a smaller compacting machine should be used to carefully compact near the edges and wing walls of a structure.

iv) As the backfilling section is liable to the inflow of rainwater or standing water, it is essential to prevent the inflow of rainwater as much as possible during the work. Seepage water should preferably be dealt with by introducing a subsurface drainage ditch(es).

v) A structure must not be subject to earth pressure by backfilling or an embankment before it has reached the level of sufficient strength as structure. Even after a structure has reached such a level, it should not be subject to uneven pressure. For example, backfilling for a culvert or a nearby embankment should be

evenly compacted in thin layers at a time from both sides of a structure to avoid an uneven load on either side.

Table 4-7-1 Particle Size and Properties of Suitable Materials for Backfilling

Item	Range
Maximum particle size	100 mm or less
4.760μm sieve through	25 – 100%
74μm sieve through	0 – 25%
Plasticity index (for 420μ sieve through)	10 or less

(3) Backfilling of Retaining Wall

In principle, the same material and same compacting machine as those used for the subgrade as well as filled up ground should be used for backfilling a retaining wall at the banking section while heeding the important points explained in (2) above. While drainage work is required at the back of a retaining wall, the suitable type must be selected in accordance with the subgrade material. Here, special attention is required to ensuring that the execution of drainage work does not make the compaction at the back of the retaining wall become insufficient.

When rolling compaction is difficult to perform at, for example, a retaining wall at the cutting section, good quality material should be used, spread so that the finishing thickness of a single layer is not more than 20 – 30 cm and fully compacted using a vibration compactor or similar machine. Special attention is also required in regard to the execution of drainage work.

When the quality of the backfilling material and shape of backfilling are clearly indicated for a concrete block retaining wall and others, such wall must be precisely executed to achieve the design requirements.

(4) Backfilling for Culvert

The simultaneous commencement of backfilling work with banking is desirable (Fig. 4-7-2-(a)). If there are any problems in terms of the procurement of material and deployment of machinery, backfilling work may be conducted prior to banking in the manner shown in Fig. 4-7-2-(b). Meanwhile, when preloading is conducted in an area with soft ground, backfilling is conducted afterwards. Cleaning may be required to remove banking material which has outflowed due to rainwater and which has deposited at the bottom of the backfilling section. At the same time, special attention is required to avoid the thick spreading of the material (Fig. 4-7-2-(c)).

In the case of a corrugated metal culvert, careful attention should be paid to it not being damaged by heavy machinery during backfilling work.

Significant settlement of the backfilling section after completion may be observed in the case of a culvert which is introduced at the boundary between cutting and banking or a culvert which crosses a road with a longitudinal slope. Common countermeasures include the introduction of weep holes for the side or wing walls or subsurface drainage facilities (net or porous pipes made of synthetic resin) along the walls at intervals of approximately 2 m.

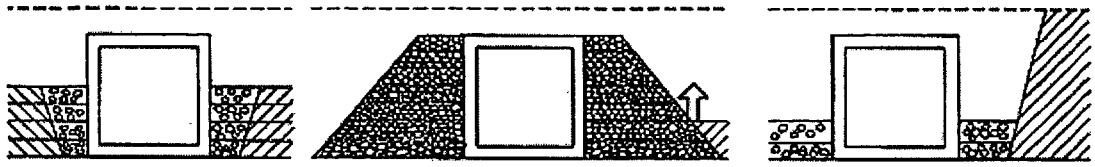


Fig. 4-7-2 Back–Fill Execution Method

(5) Approach Cushion

An approach cushion means a reinforced concrete slab positioned between such a structure as an abutment and an embankment as shown in Fig. 3-3-15 for the purpose of mitigating the adverse impacts of faulting which may occur at the connecting section between the structure and embankment. For the execution of an approach cushion, special attention should be paid to the following points.

i) The site for the approach cushion must be fully compacted.

ii) The ground at the base of the approach cushion must be carefully shaped so that it is level and that the protective covering for the reinforcing bars is not damaged.

iii) The concrete slab must be carefully cast as in the case of the concrete pavement. It must also be fully cured. As no vehicles are allowed to pass until the required strength is reached, the advance consideration of a detour is necessary.

4.8 Special Matters Relating to Execution

4.8.1 Pilot Construction

Pilot construction is conducted for many purposes, including the excavation and hauling of cohesive soil, excavation of rock, slope protection and vegetation. Here, test compaction and test banking are described.

(1) Test Compaction

1) General

As the quality of an embankment is determined by its execution method, it is desirable to conduct test compaction (model execution) in the case of the use of special banking soil or large-scale banking so that an appropriate execution method, compacting machine and work management method can be selected.

2) Important Points for Test Compaction

The soil to be used for test compaction must be the most representative material in the work section. The water content for compaction should be the natural water content and two or three different levels of water content should be selected, including near the optimal water content used for the test for soil compaction using a rammer and slightly higher water content than the optimal water content if adjustment of the water content is possible. Two or three different spreading depth should also be selected in view of the machinery to be used and the anticipated work efficiency. The number of compactions should be approximately 10 times to complete the compaction. The possible combination of machinery and suitable method for quality control should be further studied. The section for test compaction should be large enough to allow the actual execution of the intended banking.

The material tests and in-situ measurement are carried out under test compaction.

① Material Tests

Various tests are conducted to establish the characteristics of the material and reference values for compaction control. These tests include the specific gravity test for soil particles, water content test, bulk specific gravity and water absorption tests for gravel, grain size analysis test, liquid limit and plastic limit test, test for soil compaction using a rammer, cone test for soil, in-situ CBR test and crushing and slaking test for rocks if rock muck is used.

② In-Situ Measurements

Common items and the frequency of in-situ measurement are those shown in Table 4-8-1. The distribution of the measuring points should be carefully planned so that measurement can be conducted throughout the test section without hindering other measuring exercises. An example of measuring

point distribution in a measuring section is shown in Fig. 4-8-1 and an example of the test compaction results is shown in Fig. 4-8-2.

Table 4-8-1 Measuring Items and Timing of Measurement

Measuring Item	Timing of Measurement
Field density	After spreading, several times during rolling compaction (for example, at the 2nd, 4th, 8th and 10th compaction) and after rolling compaction
Water content	After spreading and after rolling compaction (can be measured during rolling compaction)
Surface settlement	After rolling compaction without fail; similar frequency as in the case of field density if possible
In-situ strength	Similar frequency as in the case of field density
Degree of deflection	As required
Plate load test, etc.	As required

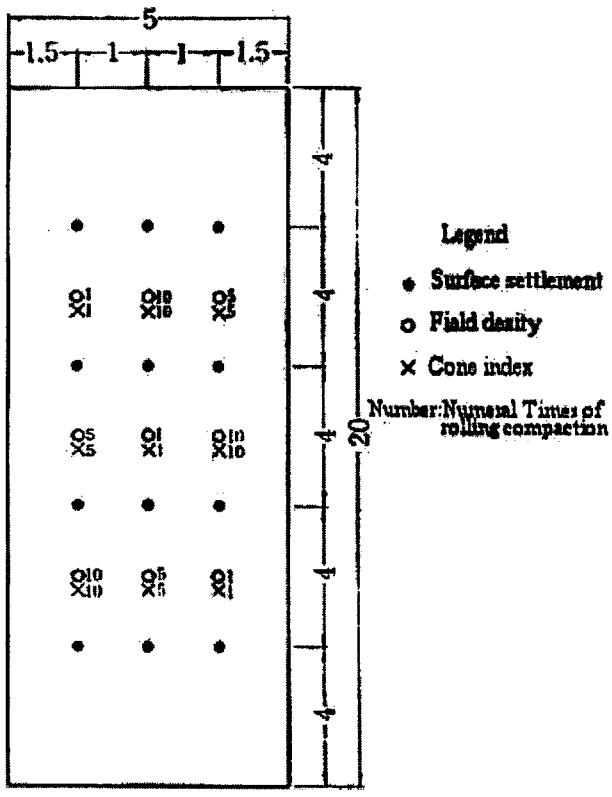


Fig. 4-8-1 Example of Measuring Point Distribution in Measuring Section

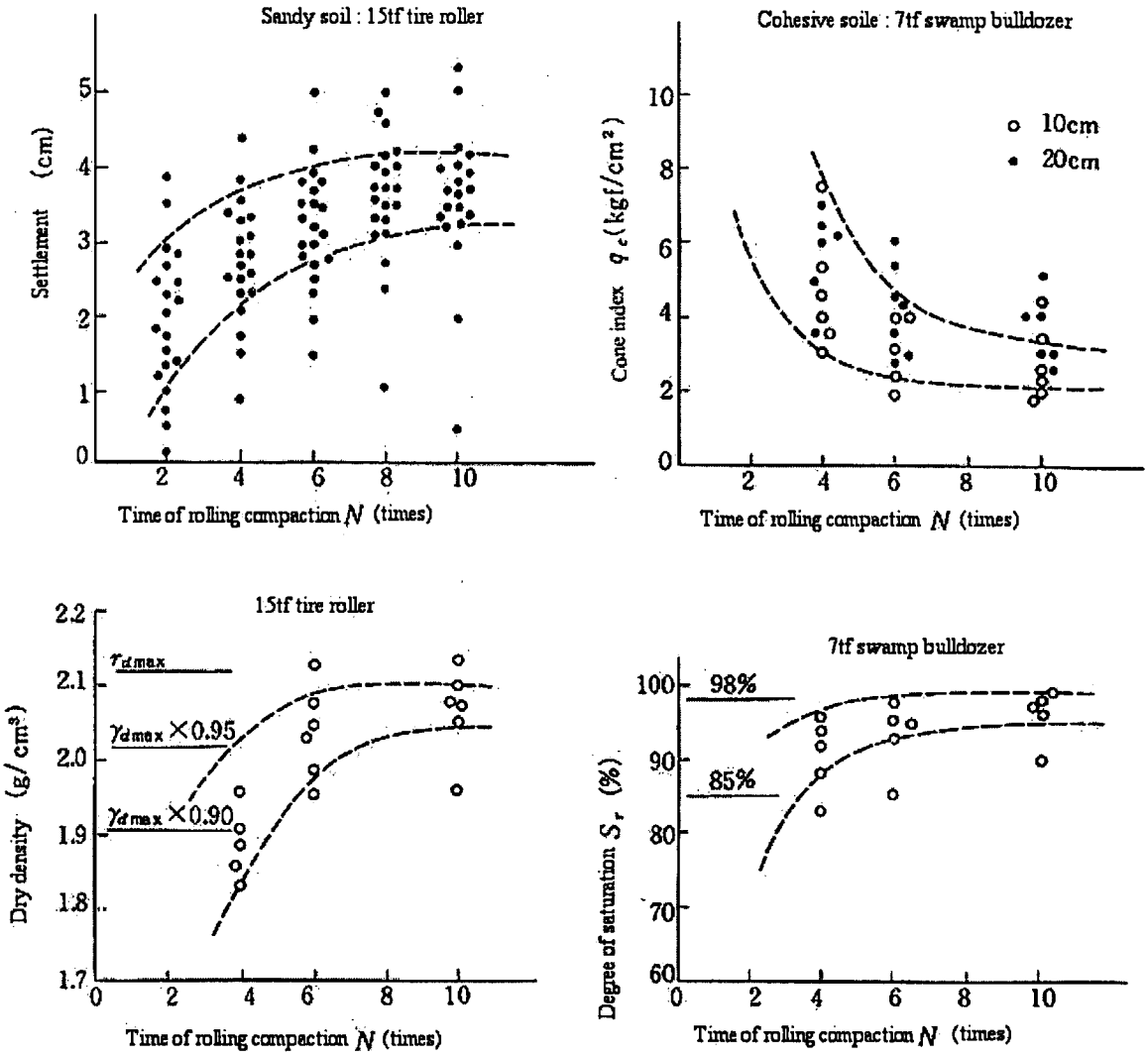


Fig. 4-8-2 Reporting of Compaction Test Result (Example)

(2) Test Banking

1) General

Test banking is conducted to understand the correlation between the meteorological conditions and efficiency of work, and to examine the method to handle unusual soil and the behaviour as well as stability of the foundation ground when banking is planned above soft ground or using unusual soil. The scale of test banking is the same as the planned size of the actual banking.

It is difficult to provide a general description of test banking, the contents of which vary depending on the subject matter for elucidation. An example of test banking on soft ground prior to the main banking work is introduced here for reference purposes.

2) Test Banking on Soft Ground

The test banking in question was conducted to study such problems of a road embankment constructed on ground mainly consisting of soft organic clay as the behaviour and displacement of the embankment for the purpose of establishing guidelines for the design and execution of a road to be constructed above such ground. The main survey items are listed below.

① Analysis of the state of settlement of the foundation ground due to embankment load
- Comparison of the changes of settlement in time between, for example, a sand drain section and an unimproved section
- Comparison of the final settlement of each section
- Analysis of the pore water pressure of the foundation ground

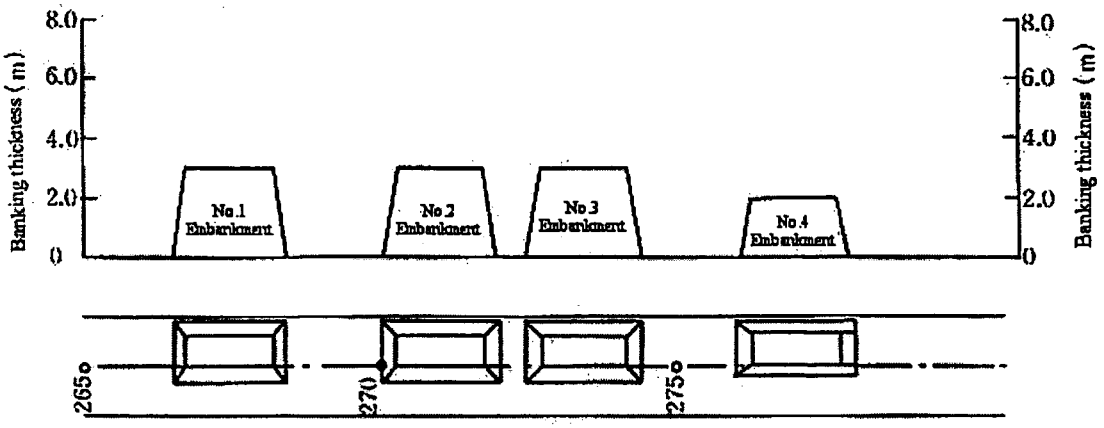
② Analysis and comparison of the embankment stability, stabilisation methods and ground improvement effects
- Trend of the shear strength increase of the ground following the progress of consolidation
- Deformation of the surrounding ground due to the construction of an embankment

③ Analysis of organic soil

④ Analysis of the execution management and efficiency of sand drain work

⑤ Comparison of the subgrade structure in connection with a low embankment

Fig. 4-8-3 shows the plan and profile of a test embankment while Fig. 4-8-4 shows the soil profile of the foundation ground of the test embankment. Meanwhile, Fig. 4-8-5 is a map showing the positioning of measuring instruments and Fig. 4-8-6 shows the main observation results.



No.	No. 1 Embankment	No. 2 Embankment	No. 3 Embankment	No. 4 Embankment
Banking Thickness	3.0 m	3.0 m	3.0 m	2.0 m
Analysis Items	Banking method	Banking method	Banking method	Subgrade construction method
Ground Stabilization	Polymer sheet (300 #) Sand mat (t = 0.7 m)	Polymer sheet (300 #) Sand mat (t = 0.7 m)	Polymer sheet (300 #) Sand mat (t = 0.7 m)	Polymer sheet (300 #) Sand mat (t = 0.7 m)
Ground Improvement	Unslaked lime ϕ = 400 mm, L = 7.8 m, n = 312	Sand train ϕ = 400 mm, L = 7.8 m, n = 312	No stabilization	No stabilization

Fig. 4-8-3 Plan and Profile of Test Banking

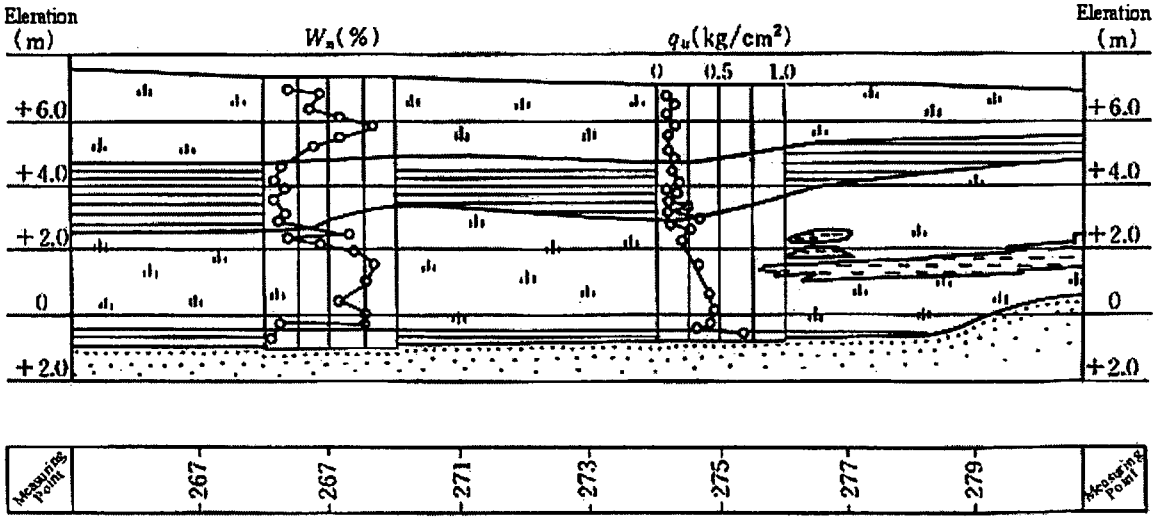


Fig. 4-8-4 Soil Profile for Test Banking Section

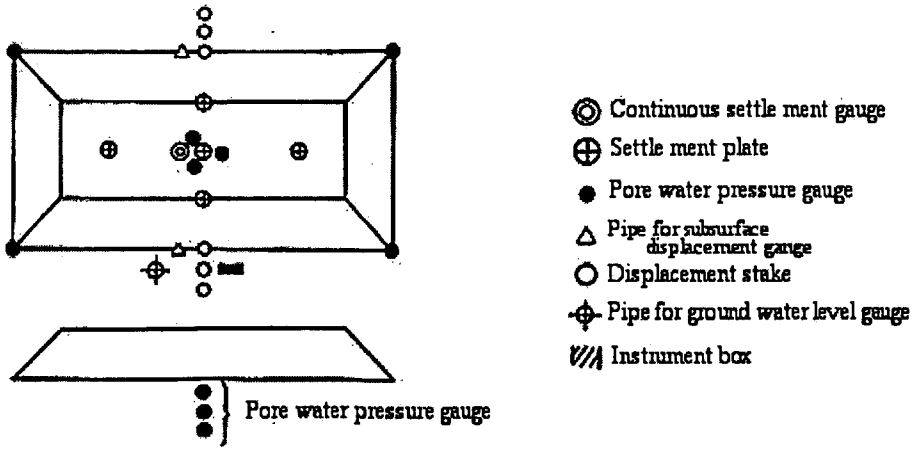


Fig. 4-8-5 Standard Deployment of Measuring Instruments

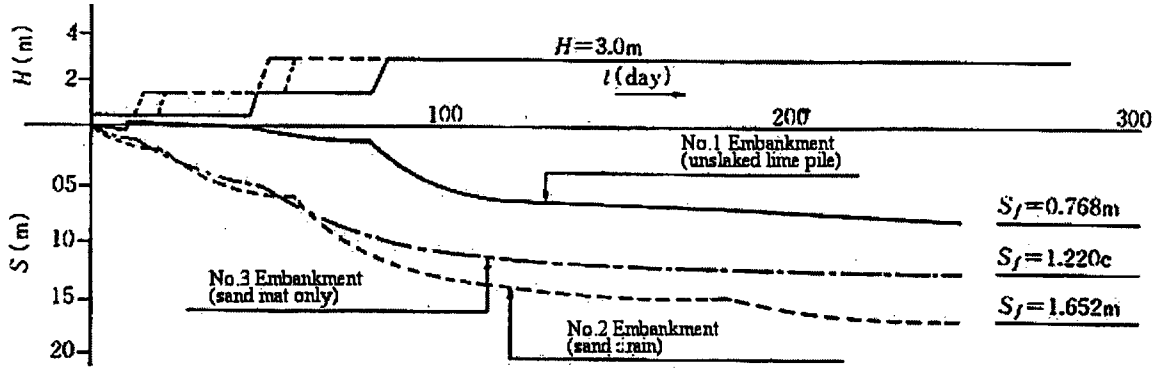


Fig. 4-8-6 Time-Settlement Curve

4.8.2 Staged Construction Work

When constructing a road, the construction of only half of the width or part of the entire cross-section sometimes occurs as the estimated traffic volume for the road has not yet reached the stage demanding the construction of the entire width, leaving the remainder for future construction in response to an increase of the traffic demand. This practice is called staged construction. A suitable construction method should be selected for staged construction depending on the situation of roadside areas, expected effects of staged construction and anticipated time until the next phase of execution. The common requirements for staged construction are listed below.

i) In general, the entire right of way is acquired at the time of original planning and, therefore, land not in service as a road must be properly preserved and managed along with improvement of the environment.

ii) When staged construction is opted for, it is desirable to construct the subgrade for the entire cross-section of the road in the case of banking. If this is found to be impossible because of the cost or other reasons, analysis of the possible impacts of the settlement of the foundation ground due to the embankment to be

constructed in the future is essential. In the case of cutting, analysis of the possibility of execution in the future while allowing traffic is necessary.

iii) Side ditches similar to those for the staged construction section should be constructed for land not in service as a road. The subgrade section should be given a cross slope of at least 3% towards the side ditches to prevent standing water. The introduction of temporary drainage channels should be considered for wide sections, such as intersections.

iv) When the existing ground is soft, it is recommended to conduct full banking to the subgrade plane to avoid differential settlement. When a transverse box culvert is entirely constructed, the introduction of a countermeasure, such as banking for the entire cross-section of the road around the box culvert, should be considered to avoid uneven pressure originating from different embankment heights. If such construction for the entire cross-section of the road is impossible, the possible impacts of future construction for the residual cross section should be analysed with a view to making the pavement and water channels temporary.

v) Construction of the substructure for the entire cross-section of the road is desirable. However, even if this is impossible, the foundation piles or footings should at least be constructed for structures of which the future construction is not easy because of their proximity to the road in service. Even in the worst case, the piles for the roadside sections of structures should be driven into the ground to facilitate the future construction of structures.

5. Work Management and Inspection

5.1 Introduction

Road work must be properly managed so that the planned and design structures with specific work specifications can be completed at a reasonable cost within a predetermined period while satisfying the required shape, dimensions and quality. Work inspection must be conducted to confirm that the completed structures meet the initial plan and design.

Two types of management are involved in work management. One is the management of "end factors" (schedule, quality and cost) relating to the activity functions of the body which directly conducts the work and the other is the management of "mean factors" as control factors to ensure the smooth performance of the said activity functions. The former is generally called execution management while the latter is called site management.

Fig. 5-1-1 shows the correlation between management and inspection.

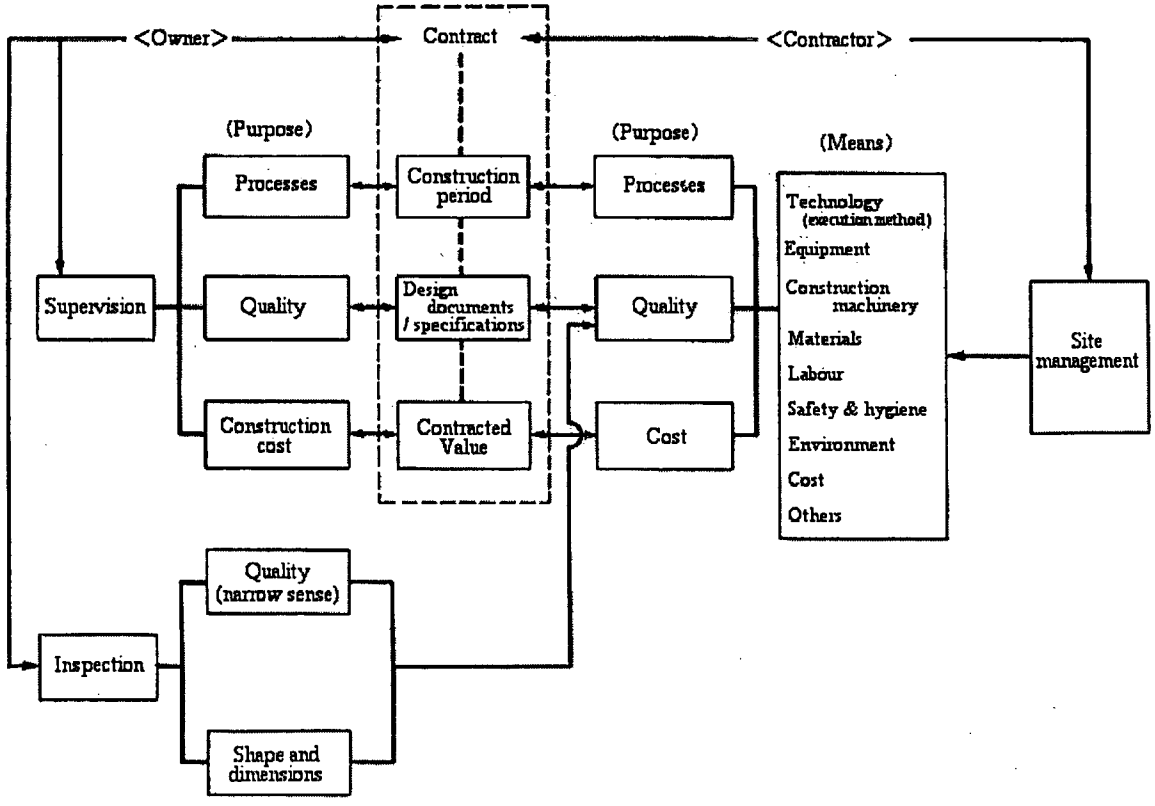


Fig. 5-1-1 Correlation Diagramme for Control and Inspection

5.2 Execution Management

5.2.1 Schedule Control

As earthwork is easily affected by the weather and others, it is not easy to proceed with the work as initially planned. For this reason, a detailed schedule plan should be prepared in advance and the planned schedule and actual progress should be compared at each stage of the work. Should any delay in the schedule occur, its cause(s) must be analysed with a view to promptly preparing a suitable countermeasure(s) as part of the strict schedule control.

5.2.2 Quality and Finished Work Control

(1) Purposes of Quality and Finished Work Control

In general, it is difficult to readjust inferior work which is discovered by inspection after completion. Even if readjustment is possible, much labour and time are required, resulting in an unfavourable economic performance. It is, therefore, rational to constantly control the quality and finished work during the construction period so that the subject items for construction pass the inspection without fail.

Precise quality control can economically complete the subject items for construction which satisfy the quality standards demanded by the specifications. The minimum dispersion of quality can enhance the reliability of construction work.

(2) Quality Control Methods

Quality control for earthwork usually centres on banking. In particular, the main control items are the banking material and degree of compaction of an embankment. Table 5-2-1 shows examples of quality control tests for embankments.

Common quality control techniques involve the execution of various tests designed to objectively evaluate the quality and statistical processing of the test results so that if there is a chance that the work quality upon completion will not meet the predetermined target(s), the necessary measures can be implemented, thereby contributing to improved quality control after these tests. In general, quality control is conducted in accordance with the steps shown in Fig. 5-2-1. The technique (control method) for the sorting of quality control data is shown in Fig. 5-2-2.

Table 5-2-1 List of Quality Control Tests (Example of Embankment)

Subject	Item	Qualitative Characteristics	Testing/Measuring Method	Remarks
Filled Up Ground	Material	Natural water content Specific gravity of soil particles Particle size Liquid limit Plasticity Limit Maximum dry density and optimal water content Cone index	Water content test for soil Specific gravity test for soil particles Particle size test for soil Liquid limit test for soil Plasticity limit test for soil Soil compaction test using a rammer Static cone penetration test	Frequency: at the beginning of the work and when the soil properties are changed
	Execution	Water content during construction Dry density Air voidage Degree of saturation Cone index	Water content test for soil In-situ unit volume weight test for soil Static cone penetration test	After rain or when a change of the water content is observed At the rate of one test/1,000 m^3; at least three times for work involving an earth volume of less than 5,000 m^3 When the trafficability is poor
Subgrade	Material	Natural water content Specific gravity of soil particles Particle size Liquid limit Plasticity Limit Maximum dry density and optimal water content CBR	Water content test for soil Specific gravity test for soil particles Particle size test for soil Liquid limit test for soil Plasticity limit test for soil Soil compaction test using a rammer CBR test	After rain or when a change of the water content is observed At the beginning of the work and when the soil properties are changed
	Execution	Water content during construction Dry density Bearing capacity value Proof rolling Degree of deflection	Water content test for soil In-situ unit volume weight test for soil Plate load test In-situ CBR test Measuring of deflection using a Benkelman beam as set forth in the Guidelines for Asphalt Pavement	After rain or when a change of the water content is observed At the rate of one test/1,000 m^3; at least three times for work involving an earth volume of less than 5,000 m^3 At the rate of one test/40 m in length for each lane Subjecting the entire width of the entire section after finishing of the subgrade



Fig. 5-2-1 Quality Control Steps

For example, if the values measuring during banking work do not satisfy the control values, an abnormality in terms of quality must be assumed, prompting the introduction of a suitable measure to prevent its reoccurrence by analysing the cause(s) of such abnormality.

In this case, if the banking material is found to be responsible for an abnormality, the source of the material must be reviewed, followed by a measure to prevent the use of such material or at least to mix it with a good quality banking material. If the soil properties of the banking material have change, a material test must be conducted. A further check should be made in regard to the appropriateness of the type and weight of the compaction equipment, spreading thickness of the soil, water content during construction and the work standard for the number of compaction operations and the necessary measures to rectify the situation should be implemented.

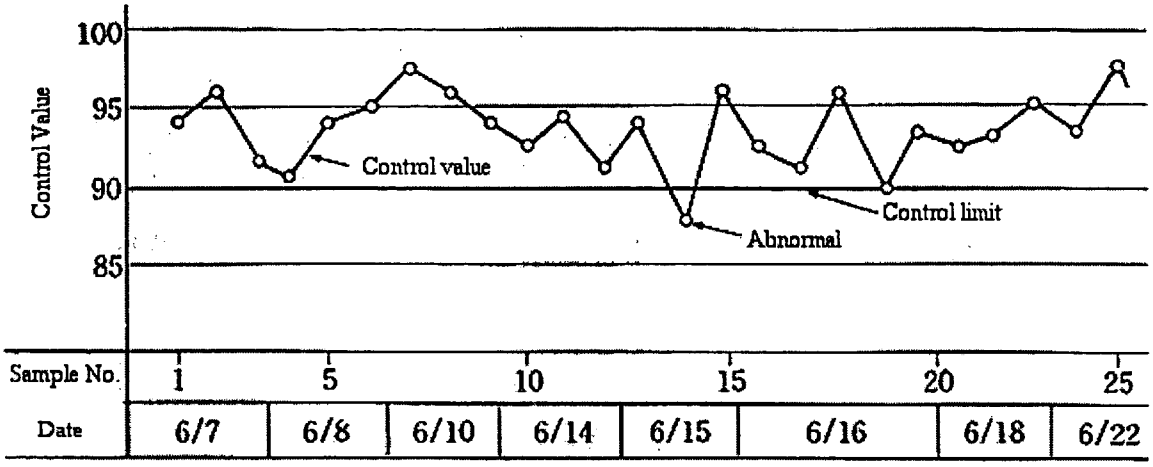


Fig. 5-2-2 Process Control Steps

(3) Finished Work Control Methods

Finished work control means to check whether or not the shape and dimensions of the subject items for construction satisfy the shape and dimensions demanded by the design documents and specifications at predetermined interim stages of the construction work in order to ensure the completion of highly reliable subject items. In the case of those parts which cannot be visually checked after the completion of construction work, photographs, etc. should be used to record the work completed.

Many finished works relating to earthworks can be visually checked after the completion of construction work. These include the position (especially the centre line of roads), dimensions (width, slope length and road length), gradient (for embankments and cut slopes) and reference height (for embankments and cut slopes). Matters requiring special attention during construction are divergence of the centre line, over-cutting of cut slopes and over-banking of embankment slopes. Careful attention should also be paid to the positioning, shape and dimensions of structures which accompany earthwork as errors cannot be easily rectified.

5.2.3 Environmental Conservation Measures

The noise and vibration caused by civil engineering work, water pollution due to the discharge of sediment, scattering of soil during transportation, dust, ground deformation by banking and water depletion by cutting, etc. can adversely affect the living environment around the work site and may also disrupt the execution of construction work. Careful attention should be paid to the planning and execution of construction work so that a suitable construction method, machinery and work procedures are selected to protect the living environment and to ensure the smooth implementation of construction work.

(1) General Environmental Conservation Measures Relating to Earthwork

i) To prevent water pollution due to the discharge of soil, a stable embankment gradient should be secured along with protection fences and others.

ii) To prevent the scattering of soil during transportation, avoidance of over-loading and the use of a sheet cover for the truck bed should be practiced together with the introduction of vehicle washing facilities at the access point to a public road from the work site.

iii) To prevent dust from the banking site, such dust prevention measures as the sprinkling of water on the embankment surface, spraying of emulsion and spraying of seeds should be conducted.

iv) To prevent water depletion due to cutting, an advance survey should be conducted to prepare a suitable prevention measure(s).

(2) Countermeasures for Noise and Vibration

The scale of the adverse impacts of noise and vibration are determined by their level, duration and frequency distribution. Viable measures include lowering of the absolute values of noise and vibration and shortening of the period in which noise and vibration are generated.

Common countermeasures for noise and vibration during construction work are listed below.

① Use of a low noise, low vibration construction method or machinery if possible
② Careful examination of the deployment positions of machinery and equipment on the site
③ Careful examination of the working hours and work processes
④ Installation of sound insulation facilities

⑤ Improved operation method of machinery

Engineers involved in construction work must fully inform nearby residents of the outline of the planned construction work to obtain their understanding and cooperation and should take the opinions of such residents into proper consideration.

5.3 Work Inspection

Inspection means to check whether or not the work has been exactly executed according to the contract at the stage of completion or partial completion (or during work progress) by means of visual observation, measurement, testing or other methods with a view to making a verdict on the pass or failure of the inspected work. Recent road work is mainly conducted by contractors and the criterion for pass is the satisfaction of the conditions specified in the contract documents (agreement, specifications and drawings, etc.) Inspection is conducted on the finished works relating to the position, height, width, length, gradient and quantity and also on the quality, such as the degree of embankment compaction.

Inspection is conducted at the completion of construction work and also during construction work.

As it is difficult to inspect all of the quantitative items involved in construction work, sampling is usually conducted and is often accompanied by a supplementary method, such as visual observation.

In special circumstances, destructive inspection is conducted for those parts which cannot be visually checked as part of the finished work and quality inspection of construction work. It is essential for any inspection to use suitable equipment to ensure reliable test or measuring results through the proper operation of such equipment.

5.3.1 Finished Work Inspection Methods

Finished work inspection involves such inspection items as the reference height, length (distance, width and slope length) and gradient in addition to confirmation of the cut slope and embankment positions. As far as measuring is concerned, it is time-consuming and not very efficient to conduct measuring at all of the measuring points. The common practice is, therefore, to conduct measuring by selecting more points at sections with many changes of the horizontal alignment as well as longitudinal alignment while selecting fewer points at sections with fewer changes, leaving the remaining points for visual checking. Visual checking is also required at the halfway points between the measuring points as the values demanded by the design drawing must be met at these points as in the case of the measuring points. If visual inspection detects any difference at a point from the preceding or

subsequent point, verification with the design drawing is required. In addition, it may also be necessary to conduct measuring in the same manner at measuring points to check for any abnormality.

(1) Inspection of Reference Height

This inspection uses a level or other suitable tool/equipment to measure the height of the centre line of the measuring point and the top of the slope and the results are compared with the corresponding heights specified in the design drawing.

(2) Inspection of Length (Distance, Width and Slope Length)

i) Inspection along Longitudinal Direction

This inspection involves measuring of the distance between measuring points, distance from a point of change of the horizontal or vertical alignment, distance between a structure and a measuring point and distance between a point of change of a cut slope or embankment and a measuring point and the results are compared with the corresponding distances specified in the design drawing.

ii) Inspection along Cross-Sectional Direction

This inspection involves measuring of the width, distance from the centre line, width of berm(s), slope length and height of the road surface at measuring points and the results are compared with the corresponding values specified in the design drawing.

(3) Inspection of Gradient

This inspection involves measuring or calculation of the gradient of the face of a cut slope and/or embankment slope at measuring points based on the horizontal distance, vertical distance, slope length and gradient meter readings and the results are compared with the corresponding values specified in the design drawing.

5.3.2 Quality Inspection Methods

The quality inspection of road earthwork practically consists of inspection of the banking material and the degree of compaction. Incidentally, the quality of vegetation work on the face of a slope and of small concrete structures is also inspected. In this section, inspection of the banking material and the degree of compaction is described.

Quality inspection of the banking material commonly involves a judgement on whether or not the quality test results meet the quality set forth in the specifications using the construction management records. Given the extreme difficulty of rectifying the quality of the banking material when it fails inspection, it is essential to confirm the quality of the soil at the initial as well as interim stages of execution in addition to a borrow pit survey.

The practical way to conduct inspection of the degree of compaction is to make a judgement based on the quality control documents recording the measured values. However, inspection of the finishing surface of the subgrade, etc. can be conducted in the form of the necessary tests upon completion of the subgrade, etc. Special attention should be paid to the sufficiency of compaction at the road edges and near structures.

Quality inspection is commonly conducted by the owner using reference data submitted by the contractor in accordance with the specifications. In addition to this, the owner may also conduct his own quality tests as part of the quality inspection process. In any case, the reliability of the tests and measuring which provide reference data is a precondition and these tests and measuring must be properly conducted under the guidance of skilled supervisors.

5.3.3 Acceptance Judgment

There are two methods as described below to judge whether or not road earthwork-related results are acceptable.

(1) Method using Standard Values

In principle, when all of the measured values of the total inspection satisfy the standard values (within the allowable error of the design values specified in the design documents and specifications), the inspected item is judged to have passed the inspection. This method is commonly used for inspection of the external dimensions of structures.

(2) Method using Acceptance Values

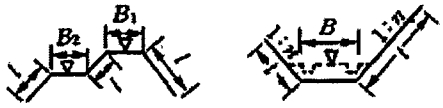
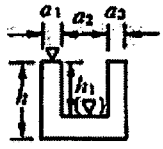
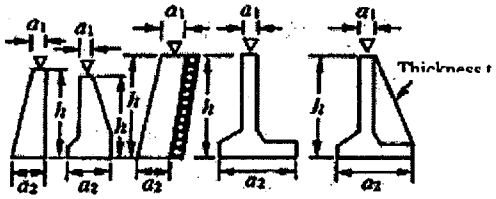
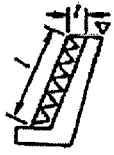
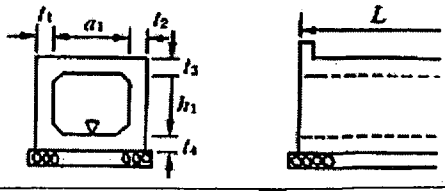
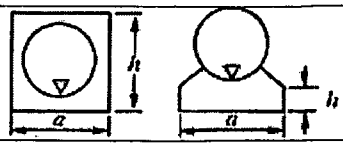
In regard to sampling inspection, when the measuring results of samples of which the quantity is determined for each lot together with the size of lot satisfy the following equation, the inspected item is judged to have passed the inspection.

Higher acceptance value $\geqq$ mean value of measured values $\geqq$ minimum acceptance value

An acceptance value is commonly set at a level where 95% of the samples meet the standard value. This method is generally used for quality inspection. Table 5-3-1 shows examples of he standard values and acceptance values used for inspection.

Table 5-3-1 Standard Value and Approval Value (Examples)

(Unit: mm)

Subject for Inspection		Standard Value	Remarks
Type of Work	Item		
Road Earthwork	Reference height	±50	
	Width B, B1, B2	-100	
	Slope length $l < 5$ m	Embankment -100 Cut slope -200	
	Slope length $l \geqq 5$ m	Embankment -2% Cut slope -4%	
	Degree of compaction (filled up ground; subgrade)		Approval value (individual measured value) (1) Maximum dry density: 85% or higher for filled up ground, 90% or higher for subgrade (2) Degree of saturation: 85 – 95% (air voidage: 10 – 2%) (3) Above the value specified in the special specifications in the case of using the strength characteristic (K value, etc.)
Concrete Site Ditch	Reference height	±30	 ▽ Side ditch
	Width a_3	-30	
	Height h, h_1	-30	
	Thickness a1, a_3	-20	
	Total length L	-200	
Concrete Retaining Wall	Reference height	±50	
	Width a_1	-30	
	Width a_2	-30	
	Height $h < 3$ m	-50	
	Height $\geqq 3$ m	-100	
	Thickness of t Counterfort	-20	
	Total length L	-200	
Stone Masonry, Block Masonry, Stone Pitching and Block Pitching	Thickness t	-50	
	Slope length $l < 3$ m	-50	
	Slope length $l \geqq 3$ m	-100	
	Reference height	±50	
	Total length L	-200	
Box Culvert	Reference height	±30	
	Thickness t_1, t_2, t_3, t_4	-20	
	Width (inner dimensions) a_1	-30	
	Height h_1	±30	
	Length $L < 20$ m	-50	
	$L \geqq 20$ m	-100	
Concrete Pipe Culvert	Reference height	±30	
	Width a	-50	
	Height h	-30	
	Total length	-200	

6. Maintenance of Earth Structures

6.1 Introduction

6.1.1 Necessity for and Components of Maintenance

The purpose of road maintenance is to ensure safe and smooth traffic through the constant preservation of the good condition as well as efficient operation of roads which are important social assets. As roads are constantly exposed to varying traffic conditions and harsh meteorological conditions, they become deteriorated or weakened in time. They may be deformed or damaged by changing conditions which are beyond anticipation at the time of their construction. Meanwhile, the social demands for roads become more advanced and diversified each year and there has been a noticeable increase of cases where defective road management is questioned, making maintenance ever more important. The maintenance, repair and disaster restoration of earth structures must be conducted against such a background.

The actual maintenance work of earth structures has the following components.

(1) Inspection

Inspection is conducted to accurately establish the conditions of earth structures, such as cut slopes and embankments, under management and their surrounding areas so that any abnormality or damage can be quickly detected.

(2) Maintenance and Repair

Maintenance or repair work is conducted to maintain or to repair the functions of earth structures.

(3) Countermeasures

Countermeasures are implemented to prevent any further worsening of abnormalities found with earth structures, or to restore the damage or failure of such structures.

6.1.2 Important Points for Maintenance

(1) Early Detection of Abnormality or Damage

The early detection of any abnormality or damage through routine inspection is important for maintenance. Early detection enables the planning of an effective countermeasure, making it easier to secure road safety.

For the effective implementation of inspection, it is essential to keep reference materials and data at the time of construction and records of subsequent maintenance for the key structural points of various road sections in an easily accessible manner.

Damage to a road and/or disruption of the traffic flow can be caused by the failure of a hillside slope or other events outside the right of way. In such cases, while those responsible for the management of areas outside the road section must principally formulate suitable countermeasures, it is desirable for those responsible for road management to pay proper attention.

(2) Prohibition or Restriction of Traffic

When an abnormality or damage is detected, it is important to implement measures to prevent traffic accidents and/or to stop the further spread of the damage. For this purpose, the prohibition or restriction of traffic must be adopted in a flexible manner in response to the extent of the abnormality or damage.

Prior to the introduction of traffic restrictions, full consideration should be given to the traffic arrangements, including the restriction method, designation of a detour and traffic safety measures. When an emergency countermeasure is implemented, careful planning must be conduced to prevent any hindrance and/or duplicated work at the time of executing full-scale restoration work.

(3) Investigation of Causes and Countermeasures

For the examination of countermeasures, the causes must firstly be accurately established. In reality, however, it may not be easy to establish the exact causes because of the overlapping of many causes or the danger associated with the on-site investigation of causes under abnormal weather conditions. Accordingly, it is necessary to urgently proceed with a viable investigation in response to the nature of abnormality or damage to formulate countermeasures, including emergency measures, in a flexible manner.

In the case of a landslide or the structural abnormality of a structure, continuous observation of the abnormality and regular investigation must be conducted.

The most important point for the formulation of countermeasures is the "disposal of water" as water is as crucial a factor for the maintenance of earth structures as during the design and execution stages.

Secondly, the key points of the planning and design of earth structures must be properly understood. Every design has its own preconditions and a proper understanding of such preconditions is essential for maintenance.

Thirdly, the characteristics and scope of application of each construction method and technology must be properly understood for the selection of a suitable countermeasure.

A flexible response must be made to the possible disruption of traffic and impacts on the roadside environment when implementing a countermeasure.

6.1.3 Disaster Restoration Measures

Japan is topographically and meteorologically prone to natural disasters. In addition to occasional record-breaking extraordinary weather, localised heavy rain is a regular occurrence. Roads in Japan are frequently damaged by rain, wind, waves, flooding and earthquakes, etc.

When roads are affected by extraordinary natural phenomena, they are often rendered impassable or road traffic is restricted. It is essential to investigate the reality of a disaster and the state of transport with a view to quickly deciding a restoration policy.

The types of disasters which frequently occur to earth structures are slope failure, embankment or retaining wall collapse due to poor drainage and the outflow of sediment due to the clogging of drainage ditches.

For the restoration of damaged earth structures, it is necessary to fully examine whether restoration of the original shape or restoration with fresh reinforcement is more desirable.

6.2 Maintenance of Slopes

The maintenance of slopes is conducted in the manner described in 6.1. The actual maintenance work consists of the planning of disaster prevention, inspection and disaster prevention maintenance work. Fig. 6-2-1 shows the general flow of maintenance work.

Disaster prevention maintenance work is conducted in response to the situation of a slope as identified by inspection. Disaster prevention maintenance work for slopes is largely classified into the maintenance management of road structures and disaster prevention management. The former involves work to repair the aged deterioration of vegetation and structures to maintain the status quo while the latter involves work to monitor abnormalities or failure and work to implement countermeasures to avert risks.

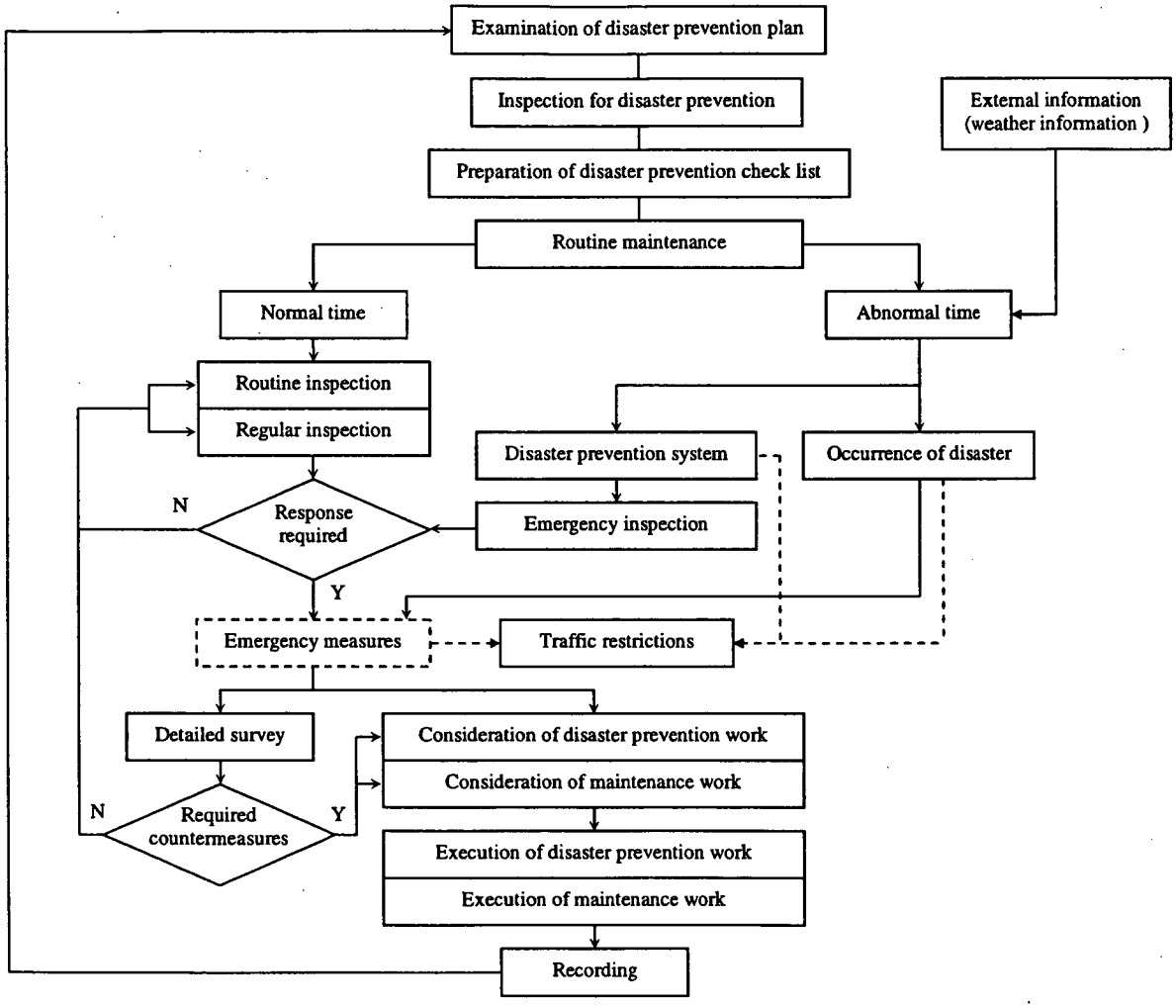


Fig. 6-2-1 Flow of Maintenance

6.2.1 Inspection of Slopes

The inspection of slopes (abnormality investigation) is conducted to maintain road safety as well as road functions. It is extremely difficult to accurately predict the level of slope stability (affected by declined strength of the ground due to weathering, improved stability due to repair or improvement of slope protection work, unpredicted external forces generated by extraordinary rainfall and earthquakes, etc., changes of land use in adjacent areas, changes of groundwater behaviour due to transformation of the topography and others) and the form of slope

disaster (travelling distance of failed soil, failure above a slope and others) on a road as well as the surrounding area.

Accordingly, the abnormality investigation of a slope is quite important in the sense that it may clarify the uncertain factors described above or, to be more precise, it may accurately identify potential sites of disaster at an early stage to prevent the occurrence of a disaster or accident. This investigation is also important to efficiently and rationally conduct repair, etc. with a limited budget and other restrictions.

(1) Types of Inspection

i) Disaster Prevention Inspection

Disaster prevention inspection is a detailed inspection which is conducted by specialist engineers to check the topographical and geological conditions of a slope, the effects of existing countermeasures and the history of disasters, etc.

ii) Routine Inspection

Routine inspection involves the visual inspection of visible areas from a road patrol vehicle, etc. and intends the early detection of any abnormality. It also checks the situation of everyday road use.

iii) Periodic Inspection

Periodic inspection involves the detailed checking of a slope by approaching the subject slope on foot and is conducted at relatively long intervals, such as once a year.

iv) Special Inspection

Special inspection is conducted after an earthquake or heavy rain, etc. when ever such inspection is deemed to be necessary to supplement routine or periodic inspection.

(2) Important Points

One of the most basic issues which are important for inspection is the inspection of water. It is well-known that the scouring and failure of a slope occurs with a large quantity of surface water running down the slope in a concentrated manner or the ground or an embankment fails due to the weeping of seepage water. Because of such a risk, it is important to conduct the careful inspection of water, including the situation of water concentration at the top of a slope, the situation of clogging of drainage ditches and the position, volume and changes of spring water.

When phenomena which are believed to be signs for the large-scale failure of a slope body or the face of a slope are detected, displacement gauges and extensometers should be installed if necessary to observe abnormalities or distortions. In addition, it may be necessary to conduct a boring survey to analyse the phenomenon of failure so that a prudent judgement can be made.

Table 6-2-1 shows the important points for slope inspection by type of slope. Depending on the degree of seriousness, abnormalities or inferior sites detected by inspection may require ordinary maintenance work or more fundamental countermeasures to prevent failure through the observation of abnormalities and a geological survey as described above.

6.2.2 Maintenance and Repair of Slopes

Maintenance and repair work is essential to maintain the proper functioning of a slope. Planting in particular can only achieve its positive effects through the normal growth of plants. Careful consideration is, therefore, required in regard to the management of planting work.

Such facilities as a rockfall prevention fence and rockfall protection net require removal of the deposited soil, repair of any cut rope or net and foot protection of the loosened foundation.

6.2.3 Countermeasures for Post-Completion Abnormalities

Even if the inspection and maintenance described in 6.2.1 and 6.2.2 are properly conducted, failure can still occur at the time of extraordinary weather, making emergency countermeasures necessary.

The general principles for the implementation of countermeasures are as follows.

① Countermeasures to deal with water, such as inflow water and spring water

② Installation of protection works to deal with the failure of a shallow surface layer or recutting to make the slope gradient gentler

③ Removal of soil from the top part of a slope by excavation, prevention pile work and/or anchor work to deal with the deep failure of a cut slope

④ Creation of a counterweight embankment or reinforcement of the bottom of a slope to deal with the failure of a high embankment

Table 6-2-1 Important Points for Slope Inspection

Type of Slope/ Slope Surface / Inspection Item	Natural Slope	Rockfall Prevention Work		Rockfall Protection Work		Vegetated Slope Surface	Slope Surface with Structural Protection			
		Concrete/ Masonry Foot Protection Work	Rock Bolt/Rock Anchor/Wire Rope Work	Rockfall Prevention Net/Fence	Rockfall Protection Retaining Wall/Rock Shed		Block Masonry Cribwork, etc.*	Concrete/Mortar Spraying	Rockfall Prevention Fence/Net	Net Hurdling; Gabion
History of Failure	▪ Survey on history of failure during and after road construction ▪ Any relation between rainfall or seismic intensity and failure as well as scale of failure									
Topography Geology	Is the topography likely to gather rainwater? Is the geology liable to weathering: Is there a dip slope?					As left	As left when a slope or vegetated slope surface exists above the slope formed by a structure		Is the topography likely to gather rainwater? Is the geology liable to weathering: Is there a dip slope?	
State of Spring Water	Location and amount of spring water; change of state of spring water				-	Location and amount of spring water; change of state of spring water	Situation of dewatering pipes; change of state of spring water		Location and amount of spring water; change of state of spring water	
Drainage Ditch	-	-	-	-	-	Clogging of ditch; scouring around vertical ditch	Clogging of ditch	Clogging of ditch	Clogging of ditch	Clogging of ditch; scouring around vertical ditch
Abnormality of Slope Surface or Body	Presence of pumice and/or cracks	Presence of pumice and/or cracks; presence of failure/scouring		Presence of pumice and/or cracks	Presence of pumice and/or cracks	Presence of cracks	(see below)	(see below)	Presence of pumice and/or cracks	(see below)
Abnormality of Slope Surface or Slope Protection Work	-	Cracking or breakage of concrete; loosening or failure of masonry work; loss of backfilling material	Looseness, damage to and/or corrosion of metal parts	Corrosion or cutting of rope or net; loosened anchor; deposited sediment; damage to or corrosion of steel; storage of sediment	Settlement of foundations of rockfall protection works; corrosion or deformation of members; loss of cushioning material; sedimentation of fallen soil; cracking or breakage of concrete	State of vegetation growth	Cracking; bulging; settlement of foundations; scouring; loosened filling material; loss of backfilling material	Cracking; bulging; partial separation; cavity between slope and the ground	Corrosion or cutting of rope or net; loosened anchor; deposited sediment	State of corrosion; state of sliding or lifting

Note: * Stone pitching, concrete pitching, block pitching and retaining walls are included in this column.

(1) Slope Protection with Vegetation

i) Cut Slopes

When a sign of failure appears or when a failure takes place, its scale and scope should be quickly established. If a failure is localised with no risk of becoming a major failure for the time being, net hurdling constitutes the easiest emergency countermeasure to apply. If the scale of the failure is large, it is necessary to recut the slope to make its gradient gentler for long-term stabilisation.

As an emergency countermeasure at the time of failure, it is important to cover a cracked portion with sheeting to prevent enlargement of the damage by preventing the permeation of rainwater.

ii) Embankment Slope

Failure on an embankment slope may occur when water overflows from drainage ditches due to localised heavy rain or the poor maintenance of drainage ditches as described in 6.3, running down the slope in a highly concentrated manner. In such a case, restoration work should firstly be conducted with the full compaction of good quality soil. If necessary, net hurdling, the use of crushed stones to create a drainage conduit or the installation of gabions at the bottom of the slope should prove effective.

(2) Countermeasures for Protected Slopes by Structures

Typical countermeasures to deal with damage to concrete or mortar spraying and cribwork are described below.

i) Concrete or Mortar Spraying

In the case of slopes sprayed with concrete or mortar, the sprayed concrete or mortar may partially peel off and scatter due to weathering. In such a case, a rockfall prevention net should be applied as an emergency countermeasure. When the further deterioration of spray works requires a full-scale countermeasure, respraying should be conducted.

In the case of the progressive weathering of the ground, concrete cribworks or others may be required depending on the degree of weathering.

ii) Cribworks

A frequently observed abnormality of cribworks is uplifting and the eventual falling of cobblestones filling the cribworks onto the road surface. In such a case, a common countermeasure is the installation of a

rockfall prevention net or the injection of concrete to fix the cobblestones together with a measure to deal with spring water.

Next, when spring water is observed through sandy soil at the back of the cribworks, the caving in of the cribworks and/or loss of the filling soil could occur due to scouring. In this case, the reinstallation of the cribworks and/or foot protection work with concrete together with a measure to deal with spring water is required.

(3) Slope Stabilisation

From the viewpoint of slope stabilisation, when boulders and pumice stones are situated on the face of a slope, the first action is to consider their removal. If removal is difficult, the following countermeasures should be adopted.

Firstly, boulders and pumice stones which are likely to fall at some stage are gathered to a safe place on the slope or are fixed by concrete or anchor bolts.

Secondly, either a rockfall prevention fence or a rockfall prevention net is installed so that any rock falling unexpectedly does not pose a danger to road traffic.

Thirdly, if none of the above countermeasures are believed to be sufficient, rock fall shed to cover the entire road should be constructed.

The above countermeasures are designed to deal with dangerous boulders and pumice stones. If dangerous soil simultaneously exists, recutting to produce a slope with stable gradient or slope protection work involving a structure should be conducted in addition to the second or third countermeasure described above.

6.3 Maintenance of Drainage Facilities

In addition to routine cleaning, the maintenance of drainage facilities should include periodic inspection to regularly check whether or not these facilities are properly performing their functions. If necessary, repair or improvement work should be conducted to preserve their functions. As damage to the road surface or slopes often originates from poor drainage, the maintenance of drainage facilities is particularly important. Special attention should be paid to preventing the excessive permeation of surface water or groundwater to the filled up ground in order to avoid any adverse impacts, including damage to the pavement by softened subgrade.

6.3.1 Inspection of Drainage Facilities

Drainage facilities must be periodically inspected. An inspector should carry a drainage system map and an inspection sheet to accurately establish the conditions of each drainage facility. Inspection during or immediately after rain is effective as drainage defects tend to be more easily found. Inspection must be very carefully conducted after a typhoon and during the rainy season. The important points for inspection are listed below.

① Situation of drainage from the road surface and slope drainage facilities and situation of surface water and soil flowing onto the right of way from the surrounding area
② Situation of soil as well as boulder deposits inside and at the entrance as well as exit of transverse drainage facilities
③ Situation of water flow at the entrance and exit of subsurface drainage facilities
④ Situation of damage to drainage facilities
⑤ Situation of the connection points (catch basins and manholes, etc.) of drainage facilities and situation of terminal treatment
⑥ Situation of drainage through weep holes in the retaining wall of an embankment and tunnel, etc.
⑦ Situation of drainage facilities at the shoulders

6.3.2 Maintenance and Repair of Drainage Facilities

(1) Road Surface Drainage Facilities

In general, drainage from the road surface is extremely important to prevent a decline of the bearing capacity of the subgrade, to ensure a safe road surface for vehicle traffic and to maintain a good environment and sanitation in areas along a road.

i) Side Ditches

The periodic inspection of simply dug side ditches, U-shape side ditches and L-shape side ditches is necessary to remove any deposited fallen leaves and collapsed soil in mountain areas or rubbish in inhabited areas. Inspection should be conducted to check for any falling of the side walls or any gap at the joints between the side walls and bottom plate in the case of concrete side ditches or any damage to the joints in the case of pre-case U-shape side ditches. If any damage is found, it must be immediately repaired.

ii) Drainage Pipes

The water flow function of drainage pipes may be lost due to clogging by rubbish or soil or due to settlement of the ground. Particularly in mountain areas, boulders from the hillside or debris flow may clog a water channel and the failure of the road itself may result in such clogging, demanding special attention for the prevention or clearing of clogging.

iii) Connecting Points of Drainage Facilities

The connecting points or changing points of the cross-section of side ditches, culverts and drainage pipes tend to have a fairly complicated structure. As these points often constitute the contact points between pre-cast products and in-situ construction work, they are often marked by structural weakness. In addition, the fact that the water flow changes at these points also leads to frequent problems.

In short, special attention is required for these points as shrinkage of the cross-sectional area for water flow due to the deposition of soil and rubbish or gaps can easily occur at these points.

(2) Subsurface Drainage Facilities

Subsurface drainage facilities are created by either porous pipes or filling with a coarse material. In either case, careful attention should be paid to preventing the gathering of soil, etc. at the outlet of drainage pipes so that water can be smoothly discharged.

The structure and location of subsurface drainage facilities often become unknown with the passing of time and, therefore, it is desirable to clearly indicate their structure in the road inventory and to clearly mark their location on the spot. As it is practically impossible to conduct the routine or periodic inspection of subsurface drainage facilities except at their outlets, it is necessary to check their proper functioning by observing their run-off, etc. after rain.

(3) Slope Drainage Facilities

Most slope failures are caused by water. Rainwater can cause slope failure as it becomes surface water which erodes the top soil or seepage water which erodes a slope from inside.

Drainage ditches for the berms of a high embankment and drainage ditches constructed at the top of an embankment or cut slope must be periodically inspected with a view to removing collapsed soil, fallen rocks and weeds from these ditches so that the water collected in the ditches does not flow anywhere except in the vertical drainage ditches.

Special attention is required for pre-cast, U-shape vertical drainage ditches as the joints may separate due to differential settlement or other reasons, resulting in slope failure caused by the discharge of soil, in turn caused by scouring by water.

In such a case, the foundation material or similar should be supplemented and well compacted to reposition the ditches. Attention should also be paid to the connecting points between the drainage ditches at the top of a slope and vertical drainage ditches and any damaged areas must be immediately repaired. Any wetting or spring water at a cut slope, embankment or slope must be carefully checked and it is advisable to adopt slope gabions, lateral drainage holes or other suitable methods depending on the cause of wetting or spring water. Slope protection work, such as concrete spraying, should also be properly inspection and, if necessary, weep holes or other suitable work should be executed.

(4) Other Drainage Facilities

Other drainage facilities include those designed to drain water from the back of a retaining wall and from tunnels, underpasses, bridges and box culverts. Proper attention must be paid to inspecting and cleaning these drainage facilities as the neglect of such requirements can lead to the destruction of a structure or the failure of filled up ground. In regard to facilities to drain water from the back of a retaining wall, special attention should be paid to the situation of surface water and the drainage situation through weep holes with a view to preventing an inflow of surface water into the back of a retaining wall. Any clogged weep hole must be thoroughly cleaned. When standing water behind a structure is believed to exist in view of the amount and turbidity of leaking water through weep holes or cracks or at a location(s) of water leakage, a countermeasure, such as the introduction of additional weep holes, should be implemented depending on the situation to reduce the load of standing water acting against the structure.

6.4 Maintenance of Retaining Walls and Culverts

Although retaining walls and culverts are essentially permanent structures, they might be distorted due to external force which is beyond their design conditions caused by extraordinary weather or an earthquake. The detection of such distortion at an early stage by means of inspection is, therefore, necessary to investigate the causes and to introduce a suitable countermeasure.

The distortion of retaining walls and culverts can be caused by the settlement of the ground, lateral flow of the ground, poor drainage of surface water and groundwater and scouring of the foundation. These cause a variety of distortions, ranging from settlement and differential settlement to bulging, dislocation and cracking.

6.4.1 Inspection of Retaining Walls and Culverts

In addition to routine inspection, periodic inspection should be conducted for retaining walls and culverts. Further inspection must also be conducted at the time of extraordinary weather or an earthquake. If the timing of inspection is not right, it is difficult to determine the details of extraordinary weather and the causes of problems. It is, therefore, essential for inspection to be conducted as quickly as possible.

Another important point is to keep relevant data and information, including the original design conditions, stress calculation results, design drawings, construction records and maintenance and repair history.

The important points for the inspection of retaining walls and culverts are described in more detail below.

(1) Retaining Walls

In regard to retaining walls, special attention should be paid to the proper functioning of weep holes and the existence of residual water at the back of the wall.

As the distortion of a retaining wall tends to appear in the form of settlement or tilting as described earlier, special attention should be paid to the crown and expansion joints to check for any dislocation, opening or slippage. Peeling of the concrete, exposure of the reinforcing bars and cracks should also be checked to determine their extent, shape, distance and nature of progress, if possible.

In general, these distortions tend to be accompanied by cracks and/or settlement at the backfill part or at the back of a retaining wall and/or adjacent areas. By focusing on these irregularities, inspection can help to discover the distortion of a retaining wall and void in the backfill part.

Moreover, in the case of retaining walls which function as river bank protection, scouring of the foundation and the distortion of foot protection works should be inspected.

(2) Culverts

In the case of culverts, primary attention should be given to any faulting or cracking of the road surface as such abnormalities are frequently caused by settlement of the backfill part.

In regard to the distortion of the main body of a culvert, the trend of settlement or tilting can be established by focusing on opening or slippage. The inspection of cracks, peeling of the concrete and exposure of the reinforcing bars should be conducted in a similar manner to the inspection of retaining walls. Particular attention should be paid to the state of deformation in the case of a corrugated metal culvert or a PVC pipe culvert.

Another inspection item is the existence of suction of the backfilling material depending on the situation of water leakage into culvert.

6.4.2 Countermeasures for Distortion of Retaining Walls and Culverts

(1) Retaining Walls

When the distortion of a retaining wall occurs, the introduction of an emergency countermeasure with primary emphasis on traffic safety is preferable. Reinforcement of only the main body of a retaining wall may prove not to be a permanent solution if distortion is caused by the ground or external conditions. Careful consideration is required in regard to proper analysis of the cause(s) of distortion in order to determine whether a permanent countermeasure is required or whether an emergency countermeasure is sufficient.

Possible countermeasures for the distortion of retaining walls are listed below.

① Reinforcement of retaining wall
② Reinforcement of foundation
③ Reduction of external force acting on retaining wall
④ Renewal

i) Reinforcement of Retaining Wall

Concrete methods to reinforce a retaining wall include the injection of resin into the cracks in the case of a concrete retaining wall and restacking of the blocks in the case of the partial bulging of a concrete block retaining wall. When the bulging or cracking of a concrete block retaining wall is very severe, causing uncertainty in regard to the strength of the wall, widening of the original concrete block retaining wall with a new concrete retaining wall (Fig. 6-4-1) or anchor works (Fig. 6-4-2) are employed to reinforce the original retaining wall.

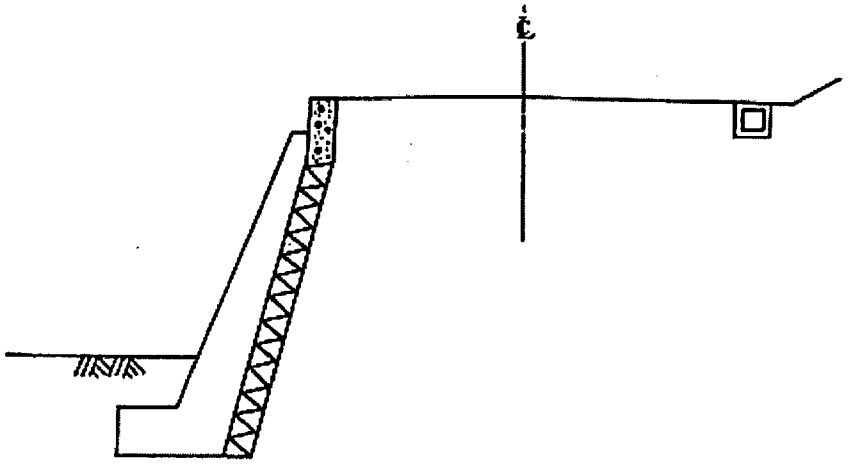


Fig. 6-4-1 Example of Reinforcing Concrete Block (Stone) Retaining Wall by Concrete Retaining Wall

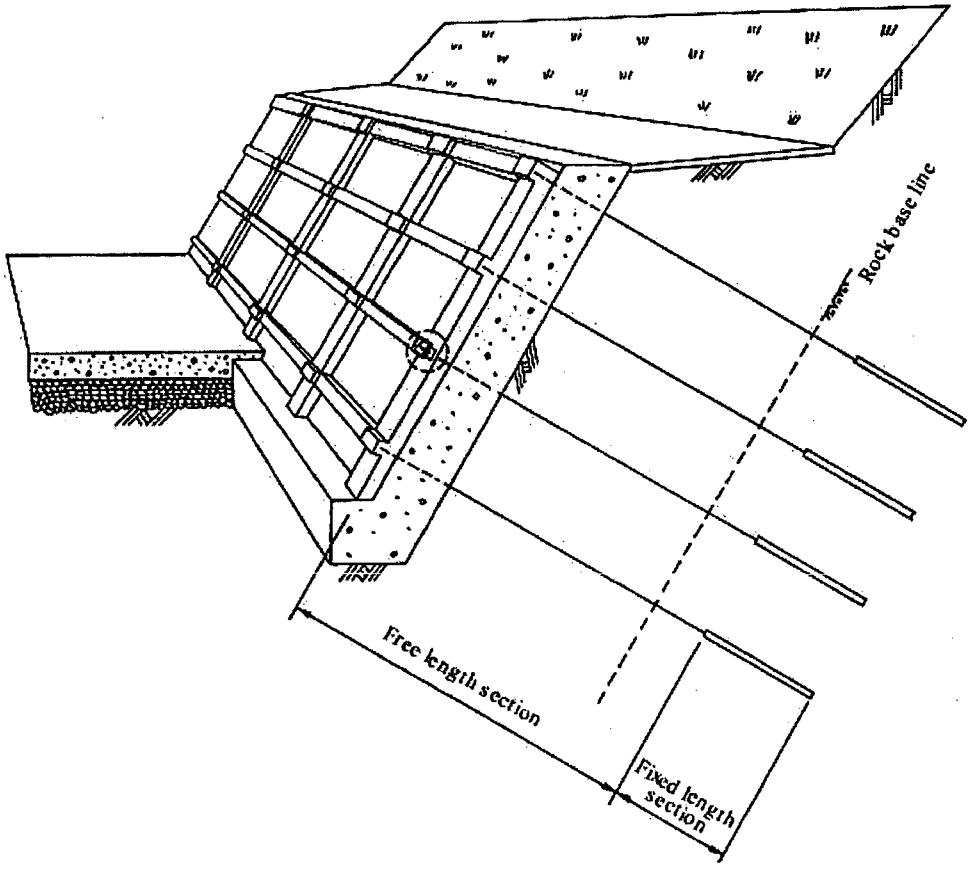


Fig. 6-4-2 Example of Anchor Works

ii) Reinforcement of Foundation

There are many foundation reinforcement methods, including (i) the use of sheet piles, steel piles, cast-in-place concrete piles or a diaphragm wall to reinforce the area around the foundation and to increase the bearing strength of the foundation and (ii) chemical injection to strengthen the ground. The most appropriate

method for the site should be adopted after proper analysis of the cause of retaining wall distortion and the conditions of the ground as well as neighbouring environment.

As all of the construction methods mentioned above involve a huge cost, it may be necessary to carefully consider whether or not a more economical method, such as counterweight fill or reduction of the external force acting on the retaining wall, may be sufficient to achieve the objective of reinforcement work.

iii) Reduction of External Force Acting on Retaining Wall

The methods to reduce the earth pressure at the back of a retaining wall are cutting (Fig. 6-4-3), prevention of the permeation of surface water to the back of a retaining wall (Fig. 6-4-4) and the cleaning of weep holes or the introduction of new weep holes.

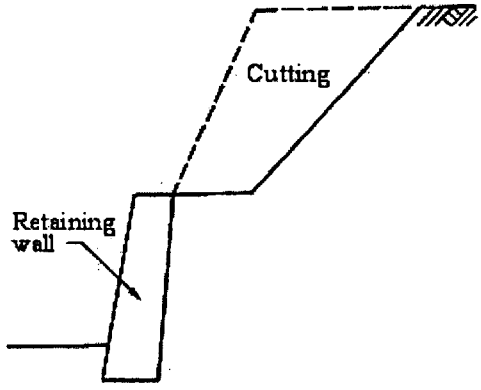


Fig. 6-4-3 Example of Reducing Back Earth Pressure by Cutting

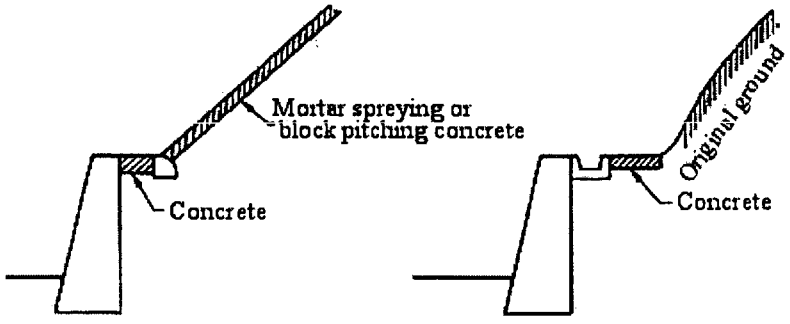


Fig. 6-4-4 Examples of Preventing Surface Water Permeation

iv) Renewal

A damaged retaining wall may be replaced by a new retaining wall. As this is a very costly fundamental countermeasure, the decision must be very carefully made. In general, the renewal of a retaining wall is required in the following cases.

① Case where the repair of a deteriorated or damaged retaining wall by means of reinforcement is impossible

② Case where sufficient reliability cannot be guaranteed despite very costly reinforcement work for the foundation

(2) Culverts

Countermeasures for the settlement of culverts include (i) lowering of approach roads in the case of an underpass and (ii) smoothing by means of lowering the water channels on both sides of a culvert by means of excavation, installation of incidental equipment (for example, installation of a drainage pump at the underpass) and raising of the water channel in the case of an irrigation canal or drainage channel.

Water leakage through the joints may be dealt with by the introduction of a conduit below the leakage point. Where the phenomenon of destruction, such as considerable cracking, which suggests the loss of the culvert function, is discovered, investigation of the causes should be conducted. If necessary, a crack gauge and other equipment should be used to determine the situation of progress so that a suitable countermeasure can be selected. The decision on any countermeasure must take the scale of cracking, original design conditions, actual state of execution and environmental conditions, etc. into consideration.

If the cracks are of a minor nature without visible progress after initial discovery, grout may be used to seal them. If further progress is detected, it may be necessary to remove the root cause or to reinforce the culvert.

6.5 Maintenance in Areas with Special Conditions

6.5.1 Countermeasures for Soft Ground

In the case of embankments constructed on soft ground, the main problem is continual differential settlement after completion rather than stability. In general, settlement continues for a long period of time and the amount of differential settlement gradually increases, making it necessary to conduct repair work at a suitable time. In particular, faulting which appears at an approach section between a structures, such as a bridge supported by piles, and an embankment poses a major problem after the commencement of service.

To properly deal with differential settlement, it is necessary to compile (i) the classification of strata, soil parameters and assumptions determined at the survey and design stages, (ii) the results of field observation during construction and other data and (iii) the locations of sites with large settlement after the commencement of service, sites where the foundation ground was severely disrupted at the time of construction work and sites where ground settlement is observed over a wide area. It is also desirable to prepare an inventory of soft ground so that an

existing reference material can be immediately used at the time of a disaster. It is also desirable to identify sites requiring special attention during maintenance patrols.

(1) Inspection

It is desirable for field observation (follow-on survey) by equipment to continue after the commencement of service at problematic sites in the past or sites with a thick layer of soft soil.

Differential settlement of the road surface can be relatively easily found by periodic patrol inspection. However, it is essential during such inspection to thoroughly check for the presence of any abnormalities, including cracks of the pavement and cracks as well as bulging of the top and face of the slope, at sites where a problem was encountered during execution and sites where substantial settlement has occurred since the commencement of service.

Careful attention should also be paid to any shifting of an abutment due to unsymmetrical earth pressure and the amount, size and changes in time of cracks in concrete structures due to differential settlement.

i) Inspection Immediately after Earthquake and during or Immediately after Rain

Inspection should be conducted, if necessary, immediately after an earthquake and during or immediately after rain to check such adverse effects of differential settlement as disruption of the drainage system, faulting and/or cracks of the road surface and abnormalities of the ground adjacent to the road and of concrete structures.

ii) Low Embankment Sites

As the foundation ground for a low embankment is considerably affected by the traffic load, the road surface at such a site often suffers from differential settlement and cracks because of differences between the soft soil thicknesses or differences between the properties of the soil constituting the soft stratum. When the road surface is not flat, the levels of noise and vibration caused by passing traffic increase, possibly causing adverse impacts on roadside areas.

iii) Approach between Structure, Such as Bridge Supported by Piles, and Embankment

The road surface of an approach between a structure supported by piles and an embankment is particularly liable to faulting because of differences in the continuing settlement after the commencement of service. Harmful voids may have been produced at such faulting of a concrete paved section or a site where an

approach cushion has been constructed. In such a case, special attention should be paid at the time of inspection to the abnormal travelling or impact sound of passing traffic.

iv) Traverse Structures and Underground Structures

In regard to a structure which is inside or which is in contact with an embankment, inspection should be conducted to check for (i) the presence and state of progress of any distortion and (ii) the extent of any impediment to the functions of such structure.

(2) Maintenance and Repair

Sites in need of repair to maintain the normal function of a road must be selected by analysing patrol inspection data, etc. It is essential to deal with sites in need of repair in an appropriate manner before the damage becomes substantial. Repair work mainly involves patching or overlay to preserve the flatness of the road surface. In addition, filling of the voids created under the approach cushion and the repair of various structures are often conducted.

i) Repair of Road Surface

Faulting due to differential settlement can cause an unpleasant feeling for road users and/or harmful impacts on road structures depending on its scale, necessitating patching or overlay.

At soft ground, there is a good chance that faulting will occur in the first five years after the commencement of service and, therefore, the need for maintenance and repair must be noted right from the beginning of the service period.

ii) Repair of Voids Created under Approach Cushion

When voids are large, they may be filled with fresh concrete, dry sand or crusher run. For this purpose, it is advisable for several filling holes to be made in the approach cushion in advance.

iii) Pulled-In Settlement of Surrounding Area Due to Embankment Load

The pulled-in settlement of the surrounding area due to the embankment load is said to extend over an area of up to double the thickness of the soft stratum. In many cases, it is extremely difficult to restore a site damaged by pulled-in settlement. When the surrounding area is cultivated land, the affected area may be replenished with borrowed material.

iv) Repair of Drainage System

When drainage of the road surface is disrupted by differential settlement, repair work must be swiftly conducted.

When a road has a median, a water channel located in the said median may suffer from damage or poor water flow due to larger settlement than that of the carriageways. As such an adverse development requires major repair work, careful attention must be paid to the construction of the median, including its construction after the execution of such a countermeasure as surcharge.

Moreover, it is desirable for the structure of a water channel to be easy to repair in view of the impacts of settlement and other adverse developments.

6.5.2 Countermeasures for Landslides

Inspection to detect any distortion of the ground and abnormalities of landslide prevention facilities is important together with the proper maintenance and repair of the said facilities. When a landslide occurs, it is particularly important to quickly commence emergency restoration work to reopen the road. When the risk of a landslide is recognised, emergency countermeasures must be urgently implemented to stabilise landslide activities to maintain road safety, subsequently followed by full-scale landslide countermeasures.

(1) Inspection

Investigation of a landslide site and its adjacent areas must be periodically conducted to visually observe the conditions of the ground and, if necessary, to measure the site to determine whether or not any distortion of the ground has taken place. Periodic inspection of landslide prevention facilities must be conducted to detect any abnormality of the facilities as soon as possible. If cracks and other ground abnormalities on the face of a slope or abnormalities of landslide prevention facilities are found, observation of the changing conditions of such abnormalities must start, followed by a survey with a view to preparing emergency countermeasures if necessary.

(2) Maintenance and Repair of Landslide Prevention Facilities

The main purpose of the maintenance of landslide prevention facilities is the blocking of the resurgence of landslide activities by preventing the functional decline of landslide prevention facilities.

There are many types of landslide prevention facilities. Typical examples are water channel works, lateral boring drainage, drainage well works, drainage tunnel works, pile works, shaft works and ground anchor works (see 3.8.3). When abnormalities of these facilities (clogging of the water channel by soil, lowering of the drainage function and distortion of the structure, etc.) are found, their causes must be quickly investigated so that any necessary repair can be conducted to restore the original functions of such facilities.

In regard to lateral boring drainage in particular, the drainage function may deteriorate after several years from original execution due to clogging of the strainers. If such clogging occurs, cleaning of the pipes with hole and strainers is recommended. In the case of lateral boring drainage which is installed to drain water from a drainage well works, the pipes with hole traversing a landslide plane may be bent or ruptured due to shearing or torsional force or their drainage function may decline due to the deposition and/or adhesion of rubbish and soil inside the pipes. To ensure the proper functioning of this type of lateral boring drainage, the state of drainage must be periodically checked and the pipes must be cleaned. In some cases, the introduction of new lateral boring drainage may be necessary.

Facilities damaged by resurgent landslides may again be destroyed if they are simply repaired. In some cases, therefore, it is necessary to implement complete restoration work or improvement work as a landslide countermeasure.

(3) Emergency Restoration of Roads

When a landslide takes place, emergency road restoration work is urgently conducted to reopen the affected road. In this case, consideration of the following is necessary to prevent damage due to a recurrence of the landslide.

i) The most desirable emergency road restoration work is the creation of a detour to avoid the area affected by the landslide. Special attention must be paid to preventing the inducement of a fresh landslide due to the construction of the detour (see Fig. 6-5-1-(a)).

ii) If it is found to be impossible to make a detour in terms of the horizontal alignment, the construction of an emergency road above the collapse soil without removing such soil deposited on the original road is desirable. This option is still valid even if the longitudinal gradient of this emergency road is steeper than before. (In this case, proper consideration should be given to drainage from the mountain side and also to any subsequent distortion of the restored road.)

iii) When the urgent restoration of a road by removing collapsed soil from the road surface is unavoidable, the employment of an earth retaining wall or similar on the mountain side is necessary to minimise the volume of soil to be removed (see Fig. 6-5-1-(b)).

iv) When a landslide occurs because of an embankment near the top of a landslide, it is necessary to remove such embankment to reduce the load at the top of the landslide as part of emergency road restoration work. For the purpose of main restoration work, alternative solutions, including a detour to an area not affected by the landslide, change of the longitudinal alignment and rehabilitation in the form of a landing bridge, must be examined.

v) In the case of a road restored by emergency countermeasures, the slope must be protected using a flexible material. At the same time, drainage facilities as well as drainage system to prevent water permeation to the landslide site must be taken into careful consideration.

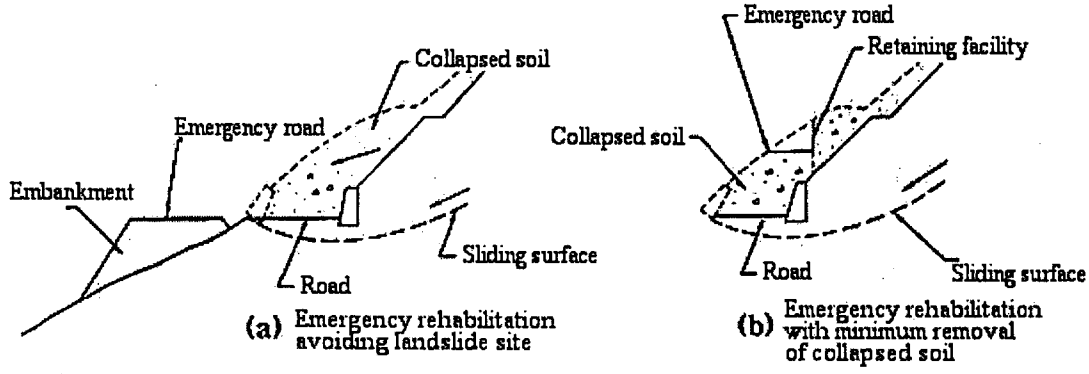


Fig. 6-5-1 Emergency Restoration at Landslide Site

(4) Countermeasures for Landslides

Countermeasures for landslides can be divided into emergency countermeasures and full-scale countermeasures

i) Landslide Survey

When a fissure and sliding on a slope are expanding, it is essential to observe the landslide speed together with analysis of the major sliding risk. The way the ground opens up is related to the scale of the landslide as well as to the soil properties. In the case of a landslide soil mass consisting of cohesive soil, sliding does not easily occur even if the strain velocity is fairly fast. In the case of weathered rock, there is a tendency to slide in a short period of time.

If any risk of a landslide is recognised, an emergency countermeasure(s) must be quickly employed together with appropriate traffic restriction measures in view of the risk posed to traffic.

ii) Emergency Countermeasures for Landslides

Emergency countermeasures for landslides are conducted to minimise the damage by slowing down the speed of a landslide and/or increasing the stability of a slope. Because of their temporary nature, those which are easy to execute at low cost are preferable.

It is desirable for the decision on the emergency countermeasure(s) to be employed to take the full-scale countermeasure(s) to be implemented in the future into consideration. Typical emergency countermeasures for landslides are described below.

① Removal of Surface Water

When a lake, pond or wet land is situated at the upper part of a slope, standing water should be drained by pumping or any other suitable means. Cracks should be covered by vinyl to prevent the permeation of water. Spring water and water from water channels or streams must be safely drained using a conduit or vinyl pipe, etc. to prevent the permeation of water into the slope.

② Removal of Groundwater

If a landslide is still active, lateral boring drainage should be constructed. In this case, the recommended length is 10 – 20 m longer over the sliding surface (see Fig. 3-8-3). Special attention must be paid to the protection of the outlet of this lateral boring drainage and the terminal treatment of the drained water.

③ Earth Retaining Wall

When there is a risk of the further spread of a surface failure which has occurred at on a steep slope at the bottom of a landslide, an earth retaining wall using gabions should be constructed at the bottom of the landslide.

④ Earth Removal Work

In the case of this type of work, the soil mass at the top of a landslide is removed by a depth of several metres (see Fig. 6-5-2). However, this type of work must be avoided if it could harm the stability of the landslide mass itself or if there is landslide topography behind the target area for site removal.

iii) Main Countermeasures for Landslides

Main countermeasures for a landslide are conducted to further enhance the stability of the ground after the stabilisation of a landslide by emergency countermeasures.

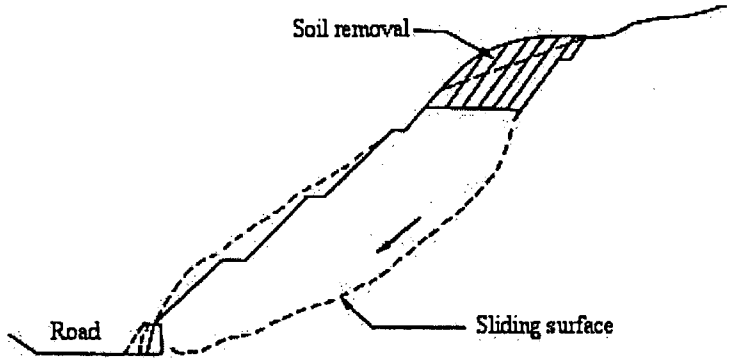


Fig. 6-5-2 Earth Removal Work (Emergency Measure)

6.5.3 Countermeasures for Debris Flow

What are important in connection with countermeasures for debris flow are (i) inspection to detect an increase of unstable soil in the upper reaches of a mountain stream and any abnormality of debris flow control facilities and (ii) the maintenance and repair of debris flow control facilities. However, when a road bridge with a sufficient cross-sectional area crosses a mountain stream, the road in question will not sustain any damage by debris flow descending the stream. Therefore, the debris flow discussed below means a debris flow which may cause or which has caused damage to a road.

When the occurrence of a debris flow is expected in view of heavy rain, the advance introduction of certain measures, including traffic restrictions, is necessary. After a disaster occurs, emergency restoration work should be conducted to re-open the road, followed by the implementation of a full-scale countermeasure(s) for debris flow. The common countermeasures for debris flow are described below.

(1) Emergency Countermeasures

When a debris flow floods the road surface, the deposited debris should be removed from the road surface as well as the removal of soil deposited at the bed of streams and in the cross-sectional area of bridges and culverts to ensure smooth water flow. The work should be carried out following the end of strong rain to avoid the outbreak of a fresh debris flow during the work. Careful attention must be paid during the countermeasure execution period to preventing secondary disasters by means of installing debris flow monitoring devices in the upstream of the work site, if necessary, in preparation for the recurrence of debris flow.

(2) Full-Scale Countermeasures

After the completion of an emergency countermeasure(s), a full-scale countermeasure(s) for debris flow should be implemented, if necessary, to prevent or alleviate damage by recurrent debris flow.

6.6 Judgement of Degree of Stability

For the prevention of road disasters due to slope failure and rockfall, etc., it is essential to judge the degree of stability of existing slopes with a view to constructing slope protection works of a higher level if necessary. The stability survey sheets used to check rockfall/slope failure, rock collapse, landslide, debris flow, embankments and retaining walls are shown in Tables 6-6-1 through 6-6-6.

At sites where the overall evaluation concludes that "a countermeasure is required" and a lengthy time is required for the implementation of a countermeasure or at sites where the overall evaluation concludes that "a disaster prevention chart should be prepared", a disaster prevention chart must be prepared and effectively used in the subsequent routine inspection. When a countermeasure is actually implemented, observation of any distortion of the slope during the execution period and arrangement of the observation results using the work execution sheet are required. An example of a disaster prevention chart and an example of a work execution sheet are shown in Table 6-6-7 and Table 6-6-8 respectively.

Table 6-6-1 Stability Survey Sheet (Rockfall and Failure)

Facility Code	

Surveyor	
Organization	

[Factors] (Ai)

Item	Factor		Cut Slope			Natural Slope		
			Scoring Criterion	Points Allotted	Score Given	Scoring Criterion	Points Allotted	Score Given
Topography	Topography with factors of failure	G1: Topographic features of talus cone G2: Trace of slope failure Clear knick line G3: The edge of plateau, Legs erosion, Overhang, Drainage slope, Trace of debris flow G4: convex slope, such as top of ridge, Overhang	Falls under G1	3		Falls under more than one type of G2	3	
			Does not fall under G1	0		Falls under one type of G2	2	
			Falls under more than one type of G2 an G3	3		No relevance to G2	0	
			Falls under one type of G2 or G3	2		Falls under more than one type of G1 and G3	3	
			No relevance to G2 or G3	0		Falls under one type of G1 or G3	2	
			Falls under G4			No relevance to G1 or G3	0	
					(6)	Falls under G4		(6)
Soil, geology and structure	Soil vulnerable to failure	▪ Soil prone to erosion ▪ Soil of which strength tends to decline with absorption of water ▪ Others	Highly noticeable	8		Highly noticeable	2	
			Noticeable	4		Noticeable	1	
			Not observed	0		Not observed	0	
					(8)			(2)
	Rock vulnerable to failure	▪ High density of cracks and weak zone ▪ Weak soft rock for erosion ▪ Rock of fast weathering, ▪ Others	Highly noticeable	12		Highly noticeable	8	
			Noticeable	6		Noticeable	4	
			Not observed	0		Not observed	0	
					(12)			(8)
	Structure vulnerable to failure	Dip slope (bedding stratification, weak line)	Observed	8		Observed	2	
			Not observed	0		Not observed	0	
		▪ Soil above impervious foundation ▪ Hard rock for upper side ▪ Weak rock for leg side ▪ Others	Highly noticeable	6		Highly noticeable	4	
			Noticeable	4		Noticeable	3	
			Not observed	0		Not observed	0	
					(14)			(6)
State of surface layer	Conditions of top soil, pumice and boulders		Unstable	12		Unstable	24	
			Slightly unstable	6		Slightly unstable	12	
			Stable	0		Stable	0	
	Unstable or slightly unstable pumice or boulders		Observed			Observed		
					(12)			(24)
	State of spring water		Observed	8		Observed	4	
			Minor seepage	4		Minor seepage	2	
			Not observed	0		Not observed	0	
					(8)			(4)

	State of surface cover	Bare to vegetation	5		Bare to vegetation (plants)	16	
		Mixed (vegetation & structure)	3		Mixed (bare, plants and trees)	10	
		Mostly covered by structure	1	(5)	Mostly covered by trees	6	(16)
Shape	Inclination (i) Height (H)	< Soil > H > 30 m	18		< Height > H ≧ 50 m	10	
		H ≦ 30, i > Standard	15		30 ≦H < 50 m	8	
		i ≦ Standard, 15 ≦ H < 30	10		15≦ H < 30 m	6	
		i ≦ Standard, H < 15	5		H < 15 m	4	(10)
		< Rock > H ≧ 50 m	18		< Gradient > i > 70°	10	
		30 ≦ H < 50 m	16		45° ≦ i < 70°	10	
		15 ≦ H < 30 m	12		i < 45°	5	
		H < 15 m	10	(18)			(10)
Abnormality	Abnormality of slope in question (fall of rocks, small rock fall, gully erosion, scour, piping, cave-in, heaving, fallen tree, crack, open crack, other strange of countermeasure work)	Plurality corresponds and clear	12		Plurality corresponds and clear	10	
		Yes or unclear	8		Yes or unclear	5	
		No	0	(12)	No	0	(10)
	Abnormality of neighbouring slope	Plurality corresponds and clear	5		Plurality corresponds and clear	4	
		Yes or unclear	3		Yes or unclear	2	
		No	0	(5)	No	0	(4)
Total Score		Cut slope:		Points (A1)	Natural slope:		Points (A2)

Notes

- The figures in brackets indicate the full score for each item.
- When any criterion is met, it should be marked with O as well as the score.
- When the observation result is unclear, an intermediate score should be given.

[Countermeasures] (Bi) = (Ai) + α or (Ai) x 0

Degree of Effect of Existing Countermeasures	Points	Score	
Anticipated rockfall or failure is fully prevented or even if it occurs, full protection is provided.	XO	Cut Slope	Slope
Anticipated rockfall or failure is failure prevented or even if it occurs, fair protection is provided but not full protection.	-20		
Anticipated rockfall or failure is partially prevented or even if it occurs, some protection is provided. However, no effect on other types of disasters.	-10		
No countermeasure has been implemented or even if one has been implemented, it has had no effect at all.	±0		
	Total Score	(B1; cut slope) Points	(B2; Slope) Points

[History] (C)

* If no rockfall or failure has occurred at the cut slope of a natural slope since the latest implementation of a countermeasure(s), evaluation based on history is unnecessary.

→ Give (C) a score of "0"

Frequency and extent of suffer	Points Allotted	Score Given
Road transport has been disrupted since the latest implementation of a countermeasure (no effect of the countermeasure)	100	
While road transport has not been disrupted, a relatively large-scale rockfall or failure reaching the road surface has been recorded (the countermeasure is not fully effective)	70	
A small-scale rockfall or failure reaching the foot of the slope has been recorded (while the existing countermeasure is not uneffective, an additional countermeasure appears to be necessary)	40	
	(C) Points	

(D) = MAX (B, C)

Score based on factors	(B) = MAX (A1, A2) Points
Score based on history	(C) Points
(B) or (C) which ever is the larger	(D) = MAX (B, C) Points

[Overall Evaluation]

Response	Judgement Result
A countermeasure is required	
A disaster prevention chart should be prepared	
No new response is required	

[Stability for Earthquake]

Stable	
Unstable	

* Mark the "Unstable" column with O if the topography category is G4 or pumice and boulders are found to be unstable.

Table 6-6-2 Stability Survey Sheet (Rock Collapse)

Facility Code	

Surveyor	
Organization	

[Factors]

Item	Factor	Scoring Criterion	Points Allotted	Score Given
Phenomenon or omen	Scale of opening or cracks	Large	30	
		Small	15	———
		None	0	(30)
	Direction of continuous lateral cracks	Dip rift	10	
		Support rift	5	———
		None	0	(10)
	Small-scale collapse/rockfall	Yes	7	———
		No	0	(7)
State of cracks	Hard rock	Regular with intervals of 1 m or more	15	
		Regular with intervals of less than 1 m	11	
		Irregular	7	———
		None	0	(15)
	Soft rock	Regular with intervals of 1 m or more	11	
		Regular with intervals of less than 1 m	7	
		Irregular	5	———
		None	0	(11)
Combination of rock properties	Hard top and soft bottom		7	
	Soft top and hard bottom		5	
	Soft throughout		2	———
	Hard throughout		0	(7)
Dip slope or support plate	Dip slope		15	
	Support plate		5	———
	None		0	(15)
Topography	Inclination (i) of slope	Overhang	4	
		$i \geqq 60°$	2	———
		$i < 60°$	0	(4)
	Height of wall	100 m or higher	10	
		50 – 100 m	7	
		30 – 50 m	4	———
		Less than 30 m	2	(10)
	Slope type	Ridge shape slope	4	
		Talus deposit slope	3	
		Valley shape slope	1	
		Intermediate slope between ridge and valley shape slope	0	——— (7)
Groundwater and rainfall	Freezing and thawing of spring water	Puddles persist for a long time or constant spring water	4	
		Frozen puddles thaw quickly or spring water after rain	2	
		Puddles do not freeze	0	——— (4)
	Icicles of spring water	Between vertical cracks	2	
		Boundaries of horizontal strata	1	———
		Hardly seen	0	(2)
		Total	(A) Points	

Notes

- The figures in brackets indicate the full score for each item.
- When any criterion is met, it should be marked with O as well as the score.
- When the observation result is unclear, an intermediate score should be given.

[Countermeasures] (B) = (A) + α or (A) x 0

Extent of effect for existing countermeasure		Score Given
Road transport has been disrupted since the latest implementation of a countermeasure (no effect of the countermeasure)	100	
While road transport has not been disrupted, a relatively large-scale rock collapse reaching the road surface has been recorded (the countermeasure is not fully effective)	70	
A small-scale rock collapse reaching the foot of the slope has been recorded (while the existing countermeasure is not uneffective, an additional countermeasure appears to be necessary)	40	
	(C) Points	

[Overall Evaluation]

Response	Judgement Result
A countermeasure is required	
A disaster prevention chart should be prepared	
No new response is required	

Table 6-6-3 Stability Survey Sheet (Landslide)

Facility Code	

Surveyor	
Organization	

[Factors] (A)

Item		Focal Matters		Points Allotted	Score Given
Landslide topography		Landslide topography, including scarp, gentle slope, disrupted contour lines or bulging towards a river, is observed	Distinctive	30	(select one)
			Slightly distinctive	15	
			Indistinctive	7	(30)
Geology and others	Geological structure, etc.	Fracture; fracture zone		18	* (select one)
		Volcanic decomposition zone; solfataric soil		18	
		Dip slope		14	
		Support plate		7	
		Massive (intrusive rock structure; cap-rock structure)		3	
		Others		0	(18)
	Period of formation and rock type	Mesozoic or paleozoic era (crystalline schist; sedimentary rock)		7	(select one)
		Tertiary (sedimentary rock)		7	
		Quaternary (unconsolidated deposits or sedimentary rock)		3	
		Others (volcanic rock; igneous rock)		0	(7)
	Spring water	Yes (including traces)		10	(select one)
		No		0	(10)
		Total (Maximum: 65)		(A) Points	

Note: Figures in brackets indicate a full score.
* When more than one focal matters are relevant, select that with the highest points for marking purposes. Mark the relevant focal matters with O even if there is more than one.

[History] (B)

Item	Focal Matters		Points Allotted	Score Given
History of landslides	Availability of reliable past data and landslide records	Yes	100	
		No	0	(100)
Landslide area	▪ Cracks upheaval or caving-in of slope ▪ Abnormality or deformation of slope stabilisation works ▪ Upheaval or cracking of road surface ▪ Minor failure (if a countermeasure is implemented after the occurrence of failure, select "no omen"	Distinctive	100	
		Slight omen	75	
		No omen	0	(100)
	Total (Maximum score: 100)		(B) Points	

(C) = MAX (A, B)

Score based on factors	(A)
Score based on history	(B)
(A) or (B) which ever is the larger	(C) = MAX (A, B)

[Countermeasures] (D) = (C) + α or (C) x 0

Degree of Effect of Countermeasure	Score (α)	Judgement
No countermeasure or insignificant effect	±0	
Certain effect	-30	
High	x 0	
Total	(D) Points	

[Overall Evaluation]

Response	Judgement Result
A countermeasure is required	
A disaster prevention chart should be prepared	
No new response is required	

* Even if no response is required, it is necessary to inspect for once or twice a year.

Table 6-6-4 Stability Survey Sheet (Debris Flow)

Surveyor	
Organization	

Facility Code	

[Factors] (A)

Item	Factor	Scoring Criterion	Points Allotted	Score Given
Characteristics of drainage basin	Source drainage basin area (drainage basin area where riverbed inclination is 15° or higher)	0.50 km^2 or larger	10	
		0.15 km^2 or larger but less than 0.50 km^2	8	
		Less than 0.15 km^2	4	(10)
	Maximum riverbed inclination	40° or more	10	
		30° or more but less than 40°	5	
		Less than 30°	0	(10)
Characteristics of slope	Area of slope face of which inclination is not less than 30°	0.20 km^2 or larger	8	
		0.08 km^2 or larger but less than 0.20 km^2	6	
		Less than 0.08 km^2	2	(8)
	Area covered by bare land or shrubs (up to 10 m in height)	0.20 km^2 or larger	8	
		0.08 km^2 or larger but less than 0.20 km^2	4	
		Less than 0.08 km^2	0	(8)
	Ongoing construction work involving unstable soil	Yes	5	
		No	0	(5)
	Presence of new cracks and/or escarp	Yes	5	
		No	0	(5)
	History of relatively large-scale failure	Yes	10	
		No	0	(10)
		Total	(A)	(56) Points

[Countermeasures] (B)

Item		Total Score for Factors (A)			
		20 or higher	15 – 19	10 – 14	Less than 10
Degree of Effect of Existing Countermeasure(s)	None or low	100	70	50	30
	Fair	70	50	30	10
	High	50	30	10	0
	Enough	0			
				(B)	Points

Notes

- The figures in brackets indicate the full score for each item.
- When any criterion is met, it should be marked with O as well as the score.
- When the observation result is unclear, an intermediate score should be given.

[Road Structure] (C) = (B) + α

Item	Scoring Criterion	Points (α)	Score Given
Road width	10 m or wider	-40	
	5 – 10 m	-30	
	3 – 5 m	-20	
	Less than 3 m	±0	
Road height	Less than 1 m or absence of bridge or box culvert	±0	
	1 – 2 m	-5	
	2 – 3 m	-15	
	3 – 5 m	-30	
	5 m or higher	-40	
	Total	(C) Points	

[History] (D)

Scoring Criterion	Points Allotted	Score Given
Traffic has been disrupted by debris flow since the latest implementation of a countermeasure	90	
Although traffic has not been disrupted, there is a history of debris flow	40	
No history of debris flow exists	0	
	(D) Points	

(E) = MAX (C, D)

Score based on factors	(C) Points
Score based on history	(D) Points
(C) or (D) which ever is the larger	(E) = MAX (C, D) Points

[Overall Evaluation]

Response	Judgement Result
A countermeasure is required	
A disaster prevention chart should be prepared	
No new response is required	

[Type of Assumed Disaster Damage]

Destruction of bridge	
Outflow of embankment	
Flooding and deposition of sediment on the road	

* Mark the relevant column with O.

Table 6-6-5 Stability Survey Sheet (Embankment)

Facility Code	

Surveyor	
Organization	

[Factors]

Factor		Scoring Criterion		Allotted Points for Each Section of Embankment					
				Half-Bank & Half-Cut Section	Both banking				Highest Score Among Various Factors
					Stream Crossing	Sloping Topography	Flat Ground	Border Between Cut Slope & Embankment	
Abnormality		Structural cracks; opening cracks		2	2	2	2	2	
		Scouring at bottom of slope		3	3	3	3	3	
		Many repaired places		2	2	2	2	2	
		Slope failure		1	1	1	1	1	___
		No abnormality		0	0	0	0	0	(3)
Foundation ground		Landslide; creep		2	2	2	2	2	
		Soft ground		1	1	1	1	1	
		Talus		1	1	1	1	1	___
		Stable ground		0	0	0	0	0	(2)
Banking material		Sandy soil		1	1	1	1	1	
		Cohesive soil		0	0	0	0	0	
		Gravelly soil		0	0	0	0	1	___
		Unknown		1	1	1	1	1	(1)
Impacts of groundwater or spring water on embankment		Wet at bottom of slope		6	6	6	6	6	
		Water channel on embankment slope		6	6	6	6	6	
		Spring water on cut slope or natural slope		6	6	6	6	6	
		Wet land use in adjacent area		-	2	2	2	-	
		No drainage ditch at mountain foot site		-	-	2	-	-	
		Insufficient cross-sectional area for side ditches and vertical drainage ditches		4	4	4	2	6	
		None		0	0	0	0	0	___ (6)
Situation of mountain stream	Current conditions of mountain stream	Debris flow and driftwood in stream		3	3	-	-	-	
		Landslide site in upstream		2	2	-	-	-	
		No constant water flow but gully is observed		2	2	-	-	-	___ (3)
		Current conditions of cross drainage facilities	Insufficient cross-section of drainage work (, D)	6	6	-	-	-	
			Insufficient treatment of end of drainage work	3	3	-	-	-	
			Bending or shrinkage of drainage work inside embankment	3	3	-	-	-	
			No cross drainage facility*	6	6	-	-	-	
			None	0	0	-	-	-	___ (6)
Impacts of riverwater or waves		Inundation at bottom of slope at time of flooding or high tide		2	2	2	2	2	
		Inundation at end of drainage works at time of flooding or high tide		2	2	2	-	-	
		Constant inundation of bottom of slope		1	1	1	2	2	

	Constant inundation of bottom of slope	1	1	1	1	1	——— (2)
	None	0	0	0	0	0	
				Total			(A) Points

Notes
- The figures in brackets indicate the full score for each item.
- When any criterion is met, it should be marked with O as well as the score.
- When the observation result is unclear, an intermediate score should be given.

* This item is only evaluated when the criterion "no constant water flow but fully is observed" is not under the "situation of mountain stream".

[Countermeasures] (b) = (A) + α

Purpose of Countermeasure	Scoring Criterion	Points Allotted	Score Given
Countermeasure for abnormality	Structural countermeasure	-4	
	Control work	-2	
	Other or none	±0	
Countermeasure for foundation ground	Ground improvement work; reinforcement of foundation	-2	
	Other or none	±0	
Countermeasure for groundwater or surface water	Groundwater drainage work; slope protection work with anchor	-4	
	Slope protection work; surface cover work	-3	
	Slope surface drainage work; vegetation cover work	-2	
	Side ditches	-1	
	Other or none	±0	
Countermeasures for mountain stream	Dam work or water stop work at valley	-5	
	Upstream/downstream channel work; earth retaining wall	-3	
	Upstream channel work	-2	
	Downstream channel work	-1	
	Other or none	±0	
Countermeasures for riverwater or waves	Earth retaining wall; bank protection work (excluding dry stone masonry)	-1	
	Other or none	±0	
	Total	(α)	(B) Points

* In the case of the score of (A) being 0, no correction of the effects of countermeasures is made.

[Score] (Score Conversion) (B) → (C)

(B)	< 0	0, 1	2, 3	4, 5	6, 7	8, 9	10, 11	12, 13	14, 15	≧ 16
(C) Points	0	10	20	30	40	50	60	70	80	90

[History] (D)

Item	Scoring Criterion	Points Allotted	Score Given
Disaster damage	Yes	+30	
	No	0	
Scale of damage	Total loss of embankment (suspension of traffic)	+70	
	Partial loss of embankment (suspension of traffic)	+60	
	Surface erosion (alternate traffic for several days)	+45	
	Minor damage (reopened on the same day)	+40	
Countermeasure(s)	Total rehabilitation of embankment; sufficient countermeasures	-70	
	???; implementation of emergency measure	-30	
	Countermeasure(s) similar to that conducted before disaster; no countermeasure	0	
	Total	(D) Points	

(E) = MAX (C, D)

Score based on factors	(C) Points
Score based on history	(D) Points
(C) or (D) which ever is the larger	(E) = MAX (C, D) Points

[Overall Evaluation]

Response	Judgement Result
A countermeasure is required	
A disaster prevention chart should be prepared	
No new response is required	

Table 6-6-6 Filling Example of Stability Survey Sheet (Retaining Wall)

Facility Code								

Surveyor	Taro Bosai
Organization	OOO Co., Ltd.

[Factors Relating to Surrounding Conditions of Retaining Wall]

Item	Factor	Scoring Criterion	Points Allotted	Score Given
Topography	Landslide	Not landslide topography	(0)	
		Landslide topography with appropriate countermeasures	5	0
		Landslide topography without countermeasures or unknown countermeasures	30	(30)
Foundation ground	Soft ground	Not soft ground	(0)	
		Soft ground with appropriate countermeasures	5	0
		Soft ground without countermeasures or unknown countermeasures	20	(20)
	Bottom of foundation	Placed on good ground	0	
		Narrow space of foundation ground in front of retaining wall	5	
		Located on talus zone	(10)	10
		Foundation ground inclines by more than 30	10	(10)
	Bearing capacity	Bearing capacity confirmed by plate load test, etc.	0	
		Bearing capacity in terms of N value, etc. estimated	2	5
		No confirmation of bearing capacity	(5)	(5)
Water	Groundwater	No spring water nearby	(0)	
		Spring water nearby	10	0
		Groundwater level in foundation ground is near bottom of retaining wall	10	(10)
	Drainage facility	Effective drainage facilities nearby prevent inflow of rainwater, etc.	0	
		Drainage facilities nearby not functioning	20	25
		Natural inflow of rainwater due to lack of drainage facilities	(25)	(25)
Site location	Scouring	No river in front	(0)	
		No scouring prevention work but foundation is above normal water level	5	
		Effective scouring prevention work in front of retaining wall	5	
		No scouring prevention work	10	0
		Scouring prevention work in front of retaining wall is ineffective	20	(20)
		Total (Maximum: 50 Points)	(A) 40 Points	

[Factors Relating to Main Body of Retaining Wall] (B)

Item	Factor	Scoring Criterion	Points Allotted	Score Given
Type of retaining wall	Stone masonry mixed	Used to protect stable ground or face of cut slope	5	
		Good backfilling work	5	
		Other than above	(10)	10
		Dry masonry	20	(20)
	No reinforcing bars	See Inspection Guidelines	5	
	Cantilever	See Inspection Guidelines	(0)	0
		Total (Maximum: 20 Points)	(B) 10 Points	

[History] (C)

Item	Factor	Scoring Criterion	Points Allotted	Score Given
Deformation of wall	No		0	
	Yes	Confirmation of no more deformation for more than two years	20	
		No progress of deformation after implementation of countermeasure (less than two years)	10	
		No progress of deformation despite absence of countermeasure (less than two years)	20	50
		No confirmation of stoppage of deformation (including no data)	(50)	(50)
		Total (Maximum: 50 Points)	(C) 50 Points	

(D) = (A) + (B) + (C)

Score based on factors relating to surrounding conditions of retaining wall	(A) 40 Points
Score based on factors relating to main body of retaining wall	(B) 10 Points
Score based on history	(C) 50 points
Total score	(D) 100 points

[Overall Evaluation]

Response	Judgement Result
A countermeasure is required	O
A disaster prevention chart should be prepared	
No new response is required	

Notes

- The figures in brackets indicate the full score for each item.
- When any criterion is met, it should be marked with O as well as the score.
- When the observation result is unclear, an intermediate sco-re should be given.

Table 6-6-7 Example of Disaster Prevention Chart

Prefecture	○ ○ Prefecture
Managed by	○ ○ Civil Engineering
Code	* * * * * * *

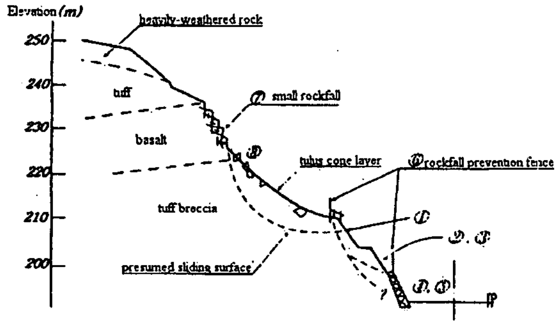
Facility Code	N * * * A 0 0 1	Inspection Issues	Rockfall and failure	Route No.	National Route No.**	Distance Markers (from)	0 0 0 0 0 0 0 0 0 0	(to)	0 0 0 0 0 0 0 0 0 0	(Up) / Down / Other	Length 120m

Operation Category	Operation / Toll	Type of Road	National road	Current or Old Road	Current road	Location	○○village ○○district	Marking	Mark at both side in red ink.	North Latitude	34''39'10.0"	East Longitude	132''11'37.0"

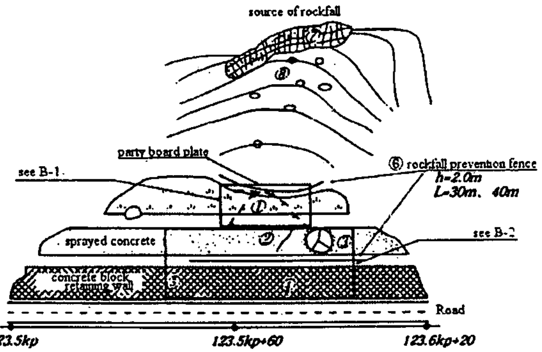
Anticipated Traffic Restrictions in Advance	(Yes) ((traffic) / Special)	Restriction Criteria	Continuous 200m time	Time - mm	Traffic Volume	Weekday	2,500vehicle / 12h	Holiday	3,800vehicle / 12h	DVD section	Yes / (No)	Bus Route	(Yes) / No	Detour	Yes / (No)

[Location Map of Inspection] ※Draw a sketch and clearly describe the location.

Cross-Section



Front Elevation



Inspection by Specialist Engineer	Yes / (No)
Abnormalities to be Checked	Key Points for Inspection

(Comments by Special Engineer)

- ○ At the slope in question, there is a possibility of rockfall from ⑦ and /or sliding failure of the slope below ⑦.
- ○ While a rockgall prevention fence is in place to deal with a small-scale rockfall, it is important to check any displacement of the concrete spray works and the concrete block retaining wall in connection with sliding failure.
- ○ The displacement of cracks on scarp ① should be measured and controlled using plate.

(1)	Countermeasure is required
2	Preparation of a chart is sufficient

Circle either 1 or 2 which ever is appropriate

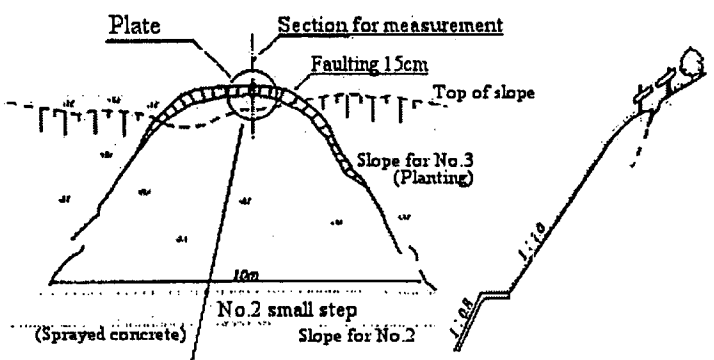
Abnormalities to be Checked	Timing of Inspection	Anticipated Type of Disasters	Responses to Abnormalities
State of cracks at scarp in upper section of sprayed concrete at point ① (see B-1) State of cracks at sprayed concrete paints ②,③ (see B-2) Bulging and spring water at concrete clock retaining wall at points ④ and ⑤ (see B-2)	○ Inspection should be conducted approximately twice a year in spring and autumn (April and Norvember). ○ Continual inspection for approx imately one week is required and also after heavy ra.n.	○ Sliding failure starting from point ① or ⑧ (Suffering ranking 2).	○ Progress of abnormality from point ① to ⑤ is observed. →Traffic restrictions and a detailed investigation by a specialist engineer should be implemented depending on the specific need.
State of rockfall prevention fence (rockfall, etc. from points ⑦and ⑧)		○ Falling or rocks (20-30cm ϕ) from ⑦ (Suffering ranking 3).	○ Deposit of fallen rocks, etc. at rockfall prevention fence at point. →Rock removal and a detailed investigation by a specialist engineer should be implemented.

Date of Preparation	1997 / 3 / 14 (weather : fine)	Name of Engineer	BOUSAI Taro	Company	OOO Corporation	Contact	Tel. OOO-OOO-OOO

Facility Code	[illegible]	Inspention Issues	Rockfall and Failure	Route Name	National Route No. **

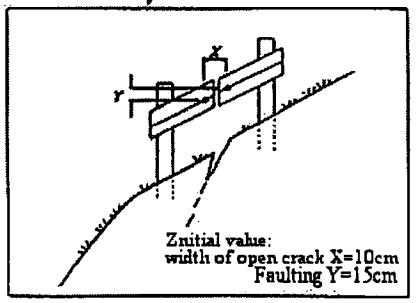
Abnormality No. ①

<Detailed Sketches>



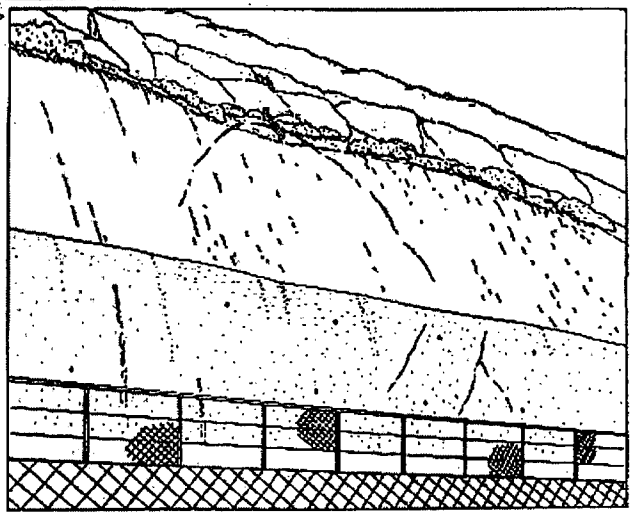
Front Elevation

Section



Enlargement

<Photograph>



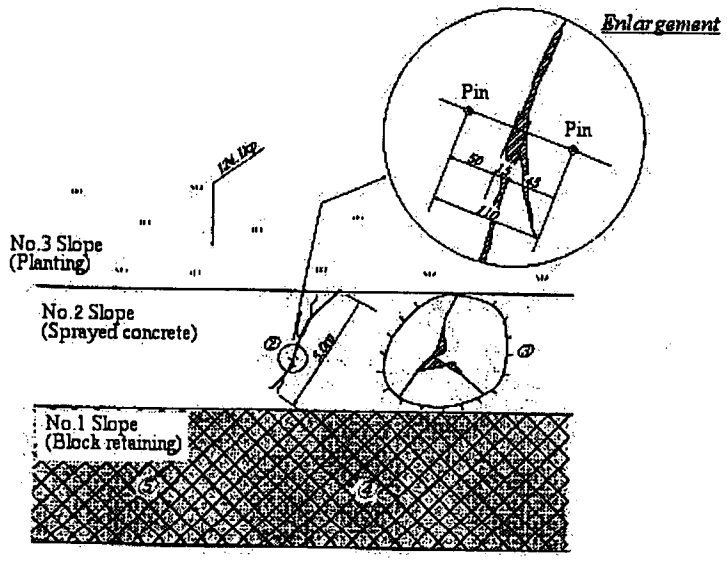
Key Point for Inspection

- O The signs of collapse have appeared at No.3 slope including top of slope, and there are slide scarps with faulting. However the tip is unclear, and it is concluded that it generated considerably before.
- O Displacement of the slide scarps in order to judge the activity during a heavy rain.
- O The plate straddling slide scarps is measurde horizontial displacement (X) and vertical displacement (Y) for two points by steal measure.
- O When an expansion tendeney is seen, the sign of slope failure is considered.

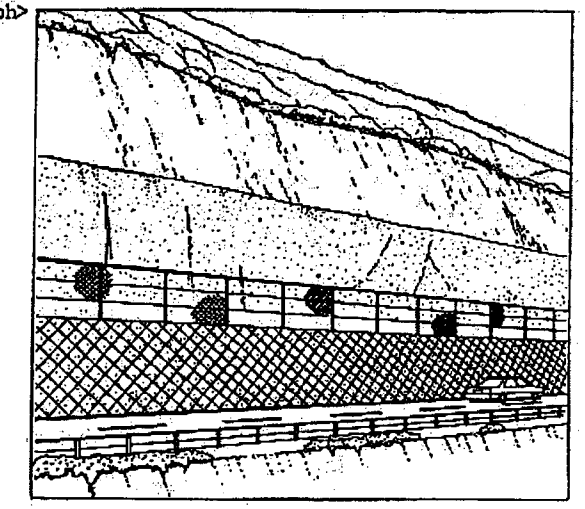
Check Points

- O Width of open crack at point ①: X (Initial value : 10cm)
- O Faulting at point ①: Y (Initial value : 15cm)

Facility Code	＊＊＊＊0001	Inspection Issues	Rockfall and Failure	Route Name	National Route No.**
Abnormality No. ②、③、④、⑤		<Detailed Sketches>			



<Photograph>



Key Points for Inspection
O Progress of cracks on sprayed concrete surface at point ②.
O State of cave-in sprayed concrete surface at point ③.
O Bulging of concrete block retaining wall at point ④.
O State of spring water through concrete block retaining wall at point ④.

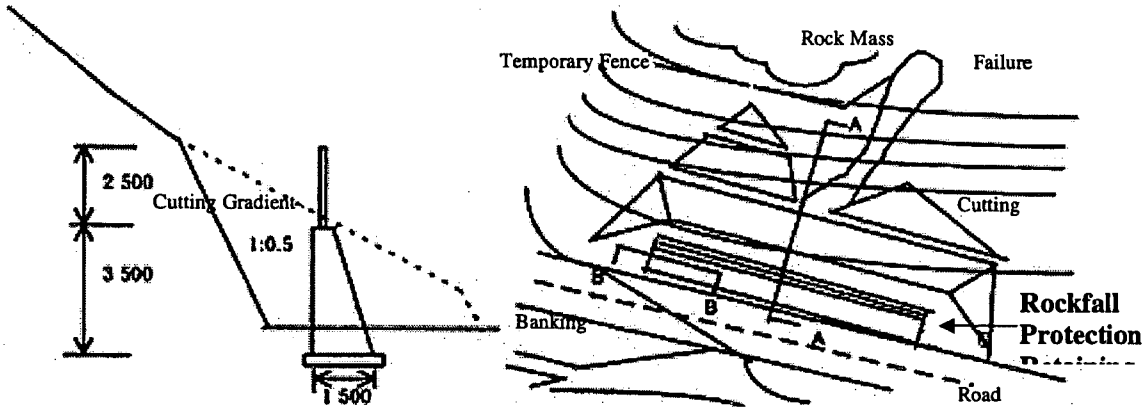
Check Points
O Cracks on sprayed concrete surface at point ② (initial values : length : 5m, width : 5m)
O Cace-in of sprayed concrete surface at point ③ : circular cave-in (minor) observed.
O Bulging of concrete clock retaining wal at point ④ (confirmed by faulting at the joint for the wall) : no abnormality at present.
O Spring water through concrete block retaining wall at point ⑤ : spring water (minor quantity) observed immediately after rain.

Facility Code	N			A	0	0	1	Inspection Issues	Rockfall/Failure	Route No.	General National Road XX	Distance Marker (from)	123.500 km (to) 123.620 km	Up/Down /Other	Length: 120 m
Inspection Date								20/04/1997	17/11/1997	01/03/1998			18/03/1998	/ /	/ /	/ /
① Cracks on No. 3 slope								Width: 10 m Faulting: 15 cm	Width: 10 m Faulting: 15 cm	Width: 10 m Faulting: 15 cm			Width: 10 m Faulting: 15 cm			
Differences from previous inspection								No change	No change	No change			No change			
② Cracks on sprayed concrete surface								Length: 5 m Width: 15 mm	Length: 5 m Width: 15 mm	Length: 5.5 m Width: 25 mm			Length: 5.5 m Width: 25 mm			
Differences from previous inspection								No change	No change	Length: +0.5 m Width: +10mm			No change			
③ Cave-in of sprayed concrete surface								Minor	Minor	-			-			
Differences from previous inspection								No change	No change	No change			No change			
④ Bulging of retaining wall								-	-	-			-			
Differences from previous inspection								No change	No change	No change			No change			
⑤ Spring water								Dripping	Dripping	Minor			Dripping			
Differences from previous inspection								No change	No change	No change			No change			
⑥ Rockfall prevention fence								-	-	Small quantity of new fallen rocks			-			
Differences from previous inspection								No change	No change	No change			No change			
⑦ State of rockfall source								-	-	-			-			
Differences from previous inspection								No change	No change	No change			No change			
⑧ State of rockfall source																
Differences from previous inspection								No change	No change	No change			No change			
Special notes at time of inspection (response at time of inspection)								Weather: fair None	Weather: cloudy None	Weather: fair ▪ Newly fallen rocks due to rain a few days ago have deposited at the fence ▪ Crack at point ② has enlarged ▪ Following measured enlargement of crack at point ②, specialist engineer is informed			Weather: cloudy ▪ No development has taken place	Weather:	Weather:	Weather:
Inspector																
Response after inspection																
(Judgement by specialist engineer)										Detailed investigation not required as observed abnormality should not lead to failure. Continued inspection recommended						
Inspection date										02/03/1998						
Engineer																

Table 6-6-8 Example of Work Execution Sheet

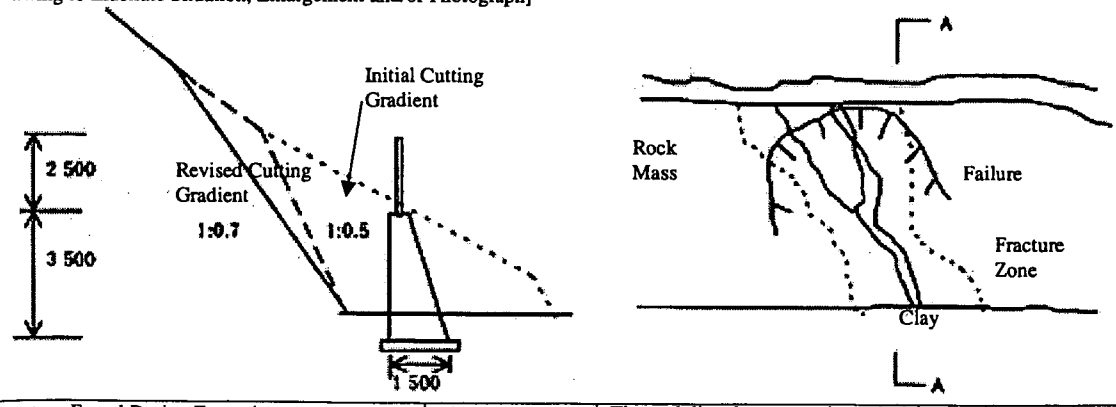
Facility Code	KXXX H001		Work Name	Disaster Prevention at			Up/Down/Other	Up
Route No.	Prefectural Route XXX		Location				Length	75 m
Distance Markers	(from) 18.625 (to) 18.700			Work Period	(from) / /H	(to / /H		
Inspection Issues for Prevention of Road Disaster		Rockfall/ Failure	Embankment	Avalanche			Year of Inspection	Heisei 8
Overall Evaluation of Inspection Results		Countermeasure Required	No Response Required	Disaster Prevention Chart			Current/Old Road	Current
Subject Load	Diameter of Fallen Rock	0.50 m	Height of Fall	25.0 m	Snow Depth	2.5 m	Other Load	None
Initial Design	Main Counter-measure	A rockfall protection retaining wall has been constructed against rockfall from above the road; cutting has been conducted to provide pockets.						
Work Cost	¥12 million		Clearly indicated important point for work execution	A temporary fence should be erected to protect workers from rockfall during the work period.				

[Design Drawing, etc.]



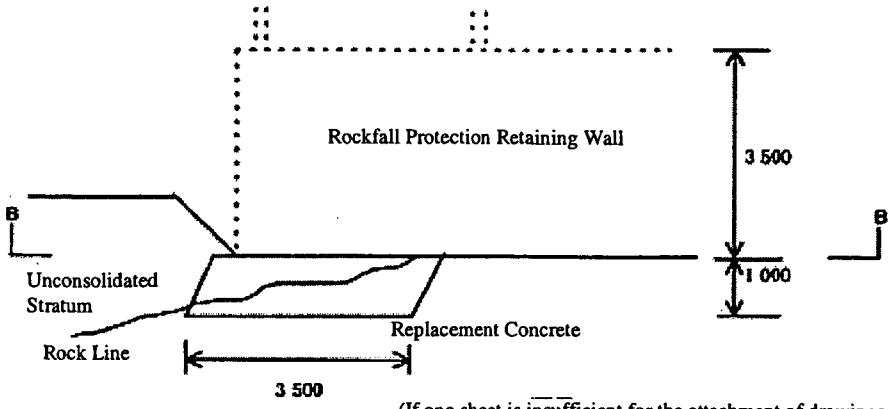
Execution Record	Outline of Abnormality, etc.	1. Failure caused by fracture zone 2. Unconsolidated stratum at the foundation	
Symptom Found During Execution		Situation	Part of the cutting face failed due to the presence of a fracture zone.
Distance Marker	18.688	Survey and Analysis	A boring survey was conducted to confirm the distribution area of the fracture zone.
Type of Work	Failure	Response	The cutting gradient was partially made gentler.

[Drawing to Illustrate Situation, Enlargement and/or Photograph]



Symptom Found During Execution		Situation	The rock line deepens at the end of the rockfall protection retaining wall, exposing the unconsolidated stratum.
Distance Marker	16.698	Survey and Analysis	Sounding was conducted to confirm the rock line.
Type of Work	Poor Foundation	Response	Replacement concrete was placed at the poor foundation section.

[Drawing to Illustrate Situation, Enlargement and/or Photograph]



(If one sheet is insufficient for the attachment of drawings, etc. use a separate sheet(s)).

Appendix

Appendix 1: List of Japanese Standards and Manuals on Highway Earthworks

Appendix 2: List of Soil Investigations and Relevant Standards

Appendix 3: List of Soil Tests and Relevant Standards

Appendix 4: Standard Rainfall Intensity Map in Japan

Appendix 5: Regional Revision Coefficient c_z for Calculation of Design Seismic Coefficient in Japan

Appendix 6: List of Reference on Highway Earthworks written in English

Appendix 7: Example of Road Damages in Earthwork Section

List of Japanese Standards and Manuals on Highway Earthworks

Name of Standard/Manual	Publisher/Editor	Year of Issue	Soil Investigation	Cut Slope	Embankment	Slope Protection	Drainage	Soft Ground	Rockfall	Landslide	Debris Flow	Retaining Wall	Culvert	Temporary Structure	Others
Manual for Highway Earthworks	Japan Road Association	1990	○	○	○	○	○	○	○	○	○	○	○	○	
Highway Earthwork Series - Manual for Survey	Japan Road Association	1986	○	△	△		△	△	△	△	△	△	△		
Highway Earthwork Series - Manual for Execution	Japan Road Association	1986		○	○										○
Highway Earthwork Series - Manual for Countermeasures for Soft Ground	Japan Road Association	1986	△					○							
Highway Earthwork Series - Manual for Drainage	Japan Road Association	1987					○								
Highway Earthwork Series - Manual for Slope Protection	Japan Road Association	1999	△	○	○	○	△		○	○	○				
Highway Earthwork Series - Manual for Retaining Wall	Japan Road Association	1999										○			
Highway Earthwork Series - Manual for Culvert	Japan Road Association	1999											○		
Highway Earthwork Series - Manual for Temporary Structur	Japan Road Association	1999												○	
Handbook of Countermeasures for Rockfall	Japan Road Association	2000							○						
Specifications for Highway Bridges Part IV; Substructures	Japan Road Association	2002													○
Specifications for Highway Bridges Part V; Seismic Design	Japan Road Association	2002													○
Design Manual Part 1 (Earthworks)	Japan Highway Public Corporation	1998	○	○	○	○		○							○
Design Manual Part 1 (Drainage)	Japan Highway Public Corporation	1990					○								
Design Manual Part 2 (Retaining Wall)	Japan Highway Public Corporation	1979										○			
Design Manual Part 2 (Culvert)	Japan Highway Public Corporation	1992											○		
Design Manual for Ground Anchor	Japan Highway Public Corporation	1992				○									
Design and Execution Manual for Reinforced Cut Slope	Japan Highway Public Corporation	1998				○									
Design and Execution Standard for Ground Anchor	Japanese Geotechnical Society	2000				○									
Method of Soil Investigations	Japanese Geotechnical Society	1995	○												
Method of Soil Test and Explanation	Japanese Geotechnical Society	1990	○												
Design and Execution Manual for Countermeasures for Liquefaction (draft)	Public Works Research Institure	1999			○			○							
Design and Execution Manual for Reinforced Earth with Geotextiles	Public Works Research Center	2000			○	△		○				○			
Design and Execution Manual for Reinforced Soil Wall (Terre Armee)	Public Works Research Center	1999										○			
Design and Execution Manual for Multi Anchor Type Reinforced Soil Wall	Public Works Research Center	1998										○			
Design and Execution Manual for Deep Mixing Method on Land	Public Works Research Center	1999						○							
Manual for Utilization Method of Soil Generated by Construction	Public Works Research Center	1997	△		△										○
Manual for River Works in Japan; Design (I)	Japan River Association	2000				△									
Manual for River Works in Japan; Design (II)	Japan River Association	2000							○	○	○				

Note: All standards and manuals above-mentioned are written in Japanese

List of Soil Investigations and Relevant Standards

Means and Purpose of Survey	Survey Method	Survey Outputs	Use of Survey Outputs	Standards/Criteria	
				Standards Code	Title of Standards
Geophysical exploration	Elastic wave exploration	Elastic wave velocity	Inference of type, properties of ground surface and state of stratification		
	Electric prospecting	Resistivity value of the ground	Particle size, porosity and water content of soil and rock		
	Velocity logging	Elastic wave velocity	Inference of Poisson's ratio, modulus of rigidity and Young's modulus	JGS 1122	Method for Seismic Velocity Logging
	Electrical logging	Resistivity value of the ground	Understanding of availability of groundwater and composition of the formations	JGS 1121	Method for Electrical Logging
Sounding	Standard penetration test	N value; collected specimens	Inference of hardness/softness of soil and soil properties, observation of specimens and various physical tests	JIS A1219	Method for Standard Penetration Test
	PWRI type cone penetration test	Number of percussions; N_d value	Survey on weathered layer at slope failure site; inference of CBR	JGS 1433	Method for Portable Dynamic Cone Penetration Test
	Portable cone penetration test	Cone penetration resistance qc	Inference of strength and other physical properties of soil; judgement on trafficability for construction machinery	JGS 1431	Method for Portable Cone Penetration Test
	Swedish weight sounding	Weight of dead weight used for penetration; number of semi-rotations per 1 m of penetration	Inference of soil strength and other physical properties of soil	JIS A 1221	Method for Swedish Weight Sounding Test
	Dutch double-tube cone penetration test	Cone penetration resistance qc	Inference of soil strength and other physical properties of soil	JIS A1220	Method for Dutch Double-Tube Cone Penetration Test
	Electric cone penetration test	Shear resistance qt; skin friction fs; pore water pressure u	Judgement on soil formation; inference of shear strength	JGS 1435	Method for Electric Cone Penetration Test
	Field vane shear test	Shear strength of soil	Calculation of shear strength of soil	JGS 1411	Method for Field Vane Shear Test
Sampling	Use of fixed piston type thin-walled tube sampler	Undisturbed soft cohesive soil	Soil test for cohesive soil, etc.	JGS 1221	Method for Obtaining Undisturbed Soil Sample Using Thin-Walled Tube Sampler with Fixed Piston
	Use of Dennison sampler	Undisturbed medium to hard cohesive soil	Soil test for cohesive soil, etc.	JGS 1222	Method for Obtaining Undisturbed Soil Sample Using Rotary Double Tube Sampler
	Use of foil sampler	Undisturbed cohesive soil	Soil test for cohesive soil, etc.	-	-
	Block sampling	Undisturbed soil	Soil test, etc.	JGS 1231	Method for Obtaining Undisturbed Soil Block Sample
Load Test	Load test inside borehole	Deformation coefficient of ground E; yield pressure q_y; limit pressure q_l	Coefficient of horizontal subgrade reaction; bearing capacity and settlement of foundations	JGS 1421	Method for Pressure Meter Test in Borehole
	Plate loading test for ground	Coefficient of ground reaction k_v; limit bearing capacity	Examination of settlement and bearing capacity of ground; inference of deformation coefficient and shear strength of ground	JGS 1521	Method for Measuring Deformability of Soil Ground by Plate Loading
	Plate loading test for roads	Coefficient of subgrade reaction k_s	Design of pavement thickness; management of compaction work	JIS A1215	Method for Plate Load Test for Soils of Roads
	In-situ CBR test	CBR	Management of subgrade and base course construction work	JIS A1222	Test Method for California Bearing Ratio (CBR) of Soils in Place

Means and Purpose of Survey	Survey Method	Survey Outputs	Use of Survey Outputs	Standards/Criteria	
				Standards Code	Title of Standards
Field Density Test	Sand replacement method	Wet density; water content; dry density	Management of compaction work	JIS A1214	Test Method for Soil Density by Sand Replacement Method
	Compacted sand replacement method	Wet density; water content; dry density	Management of compaction work	JGS 1611	Method for Measuring In-Situ Soil Density Using Compacted Sand Replacement
	Core cutter method	Wet density; water content; dry density	Management of compaction work	JGS 1613	Method for Measuring In-Situ Soil Density Using Core Cutter
	Measurement using RI	Wet density; water content; dry density	Management of compaction work	JGS 1614	Method for Measuring In-Situ Density Using Nuclear Gauge
Groundwater Survey	Measurement of groundwater level of borehole	Groundwater level	Examination of groundwater control measures	JGS 1311	Method for Measuring Groundwater Level of Borehole
	Measurement of groundwater level of observation well	Groundwater level	Understanding of long-term fluctuations of groundwater level	JGS 1312	Method for Measuring Groundwater Level of Well
	Measurement of pore water pressure of borehole	Pore water pressure	Examination of stability of soft ground against consolidation, excavation and landslide	JGS 1313	Method for Measuring Pore Water Pressure Using Electric Transducer
	Permeability test of borehole	Coefficient of permeability of ground	Examination of lowering of groundwater level and amount of spring water resulting from excavation	JGS 1314	Method for Measuring Permeability of Single Borehole
	Pumping test	Coefficient of water conveyance; coefficient of permeability; coefficient of water storage	Examination of lowering of groundwater level resulting from excavation; design of drainage method	JGS 1315	Method for Pumping Test
Field Measurement	Measurement of settlement of ground surface using settlement plate	Settlement of ground surface	Management of embankment settlement on soft ground; stability management; inference of future settlement	JGS 1712	Method for Measuring Settlement of Ground Surface Using Settlement Plate
	Measurement of displacement of ground surface using stakes	Vertical and horizontal displacement of ground surface	Management of settlement/stability of embankment on soft ground; measurement of landslide movement	JGS 1711	Method for Measuring Displacement of Soft Ground Surface Using Stakes
	Measurement of displacement of ground surface using extensometer	Displacement of ground surface	Management of settlement/stability of embankment on soft ground; measurement of landslide movement	JGS 1725	Method for Measuring Displacement of Ground Surface Using Extensometer
	Measurement of tilt of ground surface using tiltmeter	Tilting of ground surface	Understanding of landslide activity	JGS 1721	Method for Measuring Tilt of Ground Surface Using Tiltmeter
	Measurement of vertical displacement of embankment using cross arm settlement gauge	Vertical displacement of embankment	Understanding of compressive settlement of embankment and state of rolling compaction of each layer	JGS 1718	Method for Measuring Vertical Displacement of Embankment Using Cross Arm Settlement Gauge
	Measurement of horizontal ground displacement using strain gauge	Horizontal displacement of ground	Understanding of landslide plane; management of stability of embankment on soft ground	JGS 1731	Method for Measuring Ground Movement Using Strain Gauge
	Measurement of horizontal ground displacement using tiltmeter	Horizontal displacement of ground	Understanding of landslide plane; management of stability of embankment on soft ground	-	-

Note: JIS: Japanese Industrial Standard JGS: Japanese Geotechnical Standard

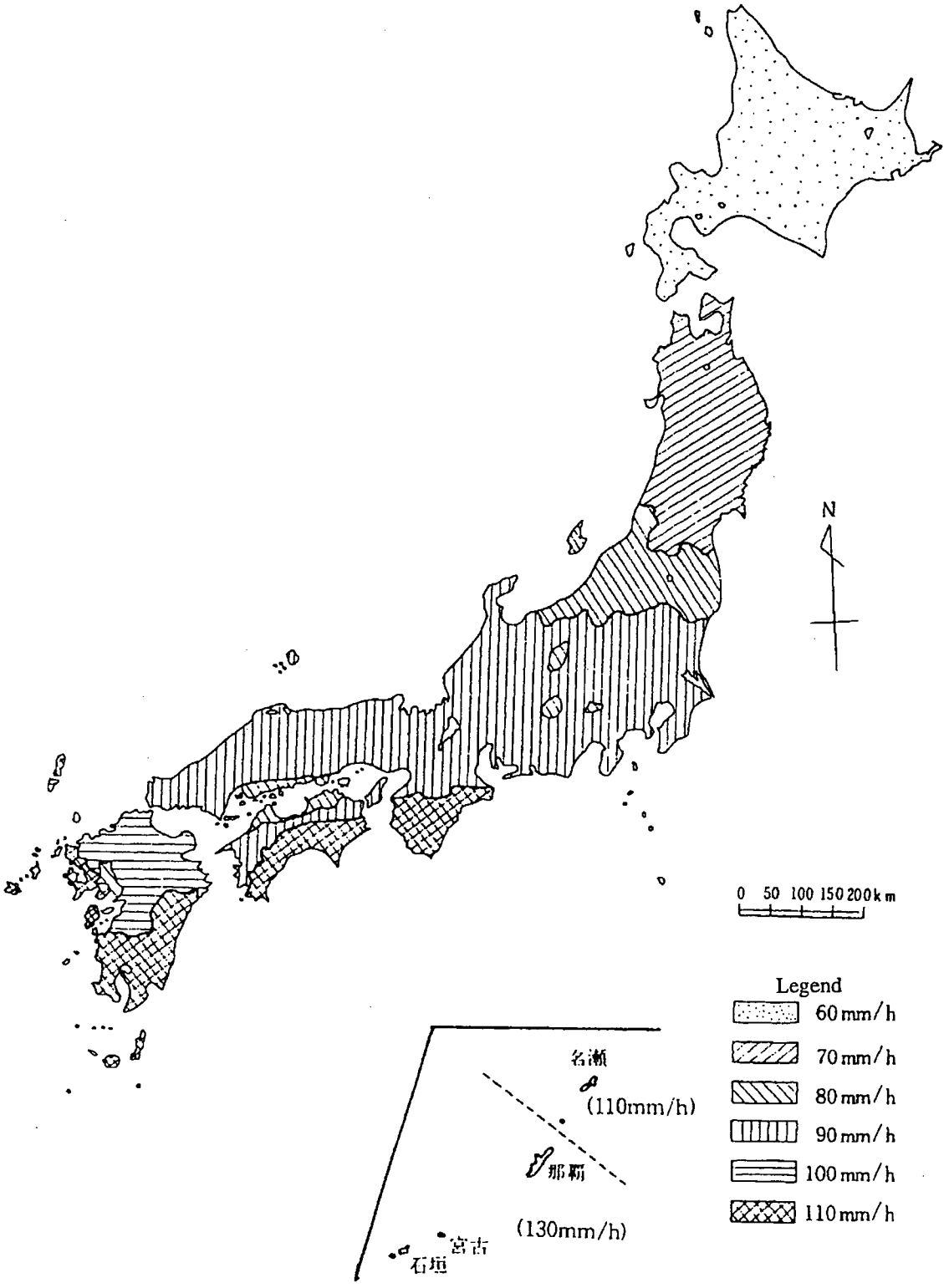
List of Soil Tests and Relevant Standards

Purpose of Test	Test Method	Test Outputs	Use of Test Outputs	Standards Code	Title of Standards
Geophysical test	Soil particle density test	Density of soil particles	Calculation of void ratio, degree of saturation, particle size, dry density and organic matter content, etc.	JIS A1202	Test Method for Density of Soil Particles
	Water content test	Water content of soil	Understanding of soil's compressibility and strength characteristics; calculation of void ratio, degree of saturation, wet density and dry density, etc.	JIS A1203	Test Method for Water Content of Soil
	Particle size test	Particle size distribution, uniformity coefficient and curvature coefficient of soil	Classification of soil; fine fraction content; judgement on permeability	JIS A1204	Test Method for Particle Size Distribution of Soil
	Fine fraction content test	Fine fraction content of soil	Soil classification of soil	JIS A1223	Test Method for Fine Fraction Content of Soil
	Liquid limit and plastic limit test	Liquid limit, plastic limit and plasticity index	Classification of fine soils; judgement on soil suitability for banking; calculation of compression index	JIS A1205	Test Method for Liquid Limit and Plastic Limit of Soil
Chemical property test	pH test	pH of soil	Examination of durability of structure, effect of ground stabilisation treatment and impacts on vegetation, etc.	JGS 0211	Test method for pH of soil
	Ignition loss test	Ignition loss of soil	Understanding of organic content and physico-chemical properties of soil	JIS A1226	Test Method for Ignition Loss of Soil
	Organic matter content test	Organic matter content of soil	Understanding of organic matter content of soil	JGS 0231	Test Method for Organic Matter Content of Soil
	Water-soluble components test	Water-soluble components content; chlorine ion content; sulphuric ion content	Examination of deterioration/durability of subsurface structure	JGS 0241	Test Method for Water-Soluble Components of Soil
Compaction test	Compaction test	Maximum dry density and optimal water content of soil	Management of compaction work	JIS A1210	Test Method for Soil Compaction Using Rammer
	Cone index test for compacted soil	Cone index of compacted soil	Judgement on trafficability during construction work	JIS A1228	Test Method for Cone Index of Compacted Soil
CBR test	CBR test	CBR	Design of pavement thickness; judgement on suitability of banking materials	JIS A1211	Test Method for California Bearing Ratio (CBR) of Soil in Laboratory
Shearing test	Unconfined compression test	Unconfined compression strength q_u; strain at failure	Examination of short-term stability of ground; strength of improved soil	JIS A1216	Method for Unconfined Compression Test of Soil
	Triaxial compression test (unconsolidated – undrained)	Cohesion c_u; shear resistance angle ϕ_u	Examination of short-term stability of ground	JGS 0521	Method for Unconsolidated – Undrained Triaxial Compression Test
	Triaxial compression test (consolidated – undrained)	Cohesion c_{cu}; shear resistance angle ϕ_{cu}	Examination of stability of embankment on soft ground	JGS 0522	Method for Consolidated – Undrained Triaxial Compression Test
	Triaxial compression test (consolidated – undrained)	Cohesion c'; shear resistance angle ϕ'	Examination of stability of embankment on soft ground (analysis of effective stress)	JGS 0523	Method for Consolidated –Undrained Triaxial Compression Test for Soil with Pore Water Pressure Measurements
	Triaxial compression test (consolidated – drained)	Cohesion c_d; shear resistance angle ϕ_d	Examination of stability of sandy ground	JGS 0524	Method for Consolidated – Drained Triaxial Compression Test
	Cyclic undrained triaxial test	Number of repeated loads	Examination of liquefaction strength of soil	JGS 0541	Method for Cyclic Undrained Triaxial Test of Soil
	Box shear test	Cohesion c; shear resistance angle ϕ	Examination of stability of earth structure	-	-

Purpose of Test	Test Method	Test Outputs	Use of Test Outputs	Standards Code	Title of Standards
Consolidation test	Consolidation test using incremental loading	Consolidation coefficient c_v; volume consolidation coefficient m_v; consolidation index C_c; consolidation yield stress p_c	Examination of consolidation settlement and speed of soft ground	JIS A1217	Test Method for One-Dimensional Consolidation Properties of Soil Using Incremental Loading
	Consolidation test using constant rate of strain loading	Consolidation coefficient c_v; volume consolidation coefficient m_v; consolidation index C_c; consolidation yield stress p_c	Examination of consolidation settlement and speed of soft ground	JIS A1227	Test Method for One-Dimensional Consolidation Properties of Soil Using Constant Rate of Strain Loading
Permeability test	Permeability test	Coefficient of permeability	Examination of stability of excavated ground and slope	JIS A1218	Test Method for Permeability of Saturated Soil
Test for soil stabilisation	Making of specimens of stabilised soil using rammer	-	Strength characteristics of improved banking materials	JGS 0811	Practice for Making and Curing Compacted Stabilised Soil Specimens Using Rammer
	Making of specimens of stabilised soil by static compaction	-	Strength characteristics of improved banking materials	JGS 0812	Practice for Making and Curing Statically Compacted Stabilised Soil Specimens
	Making of non-compacted stabilised soil specimens	-	Strength characteristics of improved ground and improved subgrade soil	JGS 0821	Practice for Making and Curing Noncompacted Stabilised Soil Specimens
	Making of specimens of stabilised soil by chemical grouting	-	Strength characteristics of improved ground	JGS 0831	Practice for Making and Curing Chemically Grouted Soil Specimens
Soil Classification	Classification of geomaterials for engineering purposes	-	Suitability of materials; planning of earth structure	JGS M111	Method of Classification of Geomaterials for Engineering Purposes

Note: JIS: Japanese Industrial Standard JGS: Japanese Geotechnical Standard

Appendix 3-2

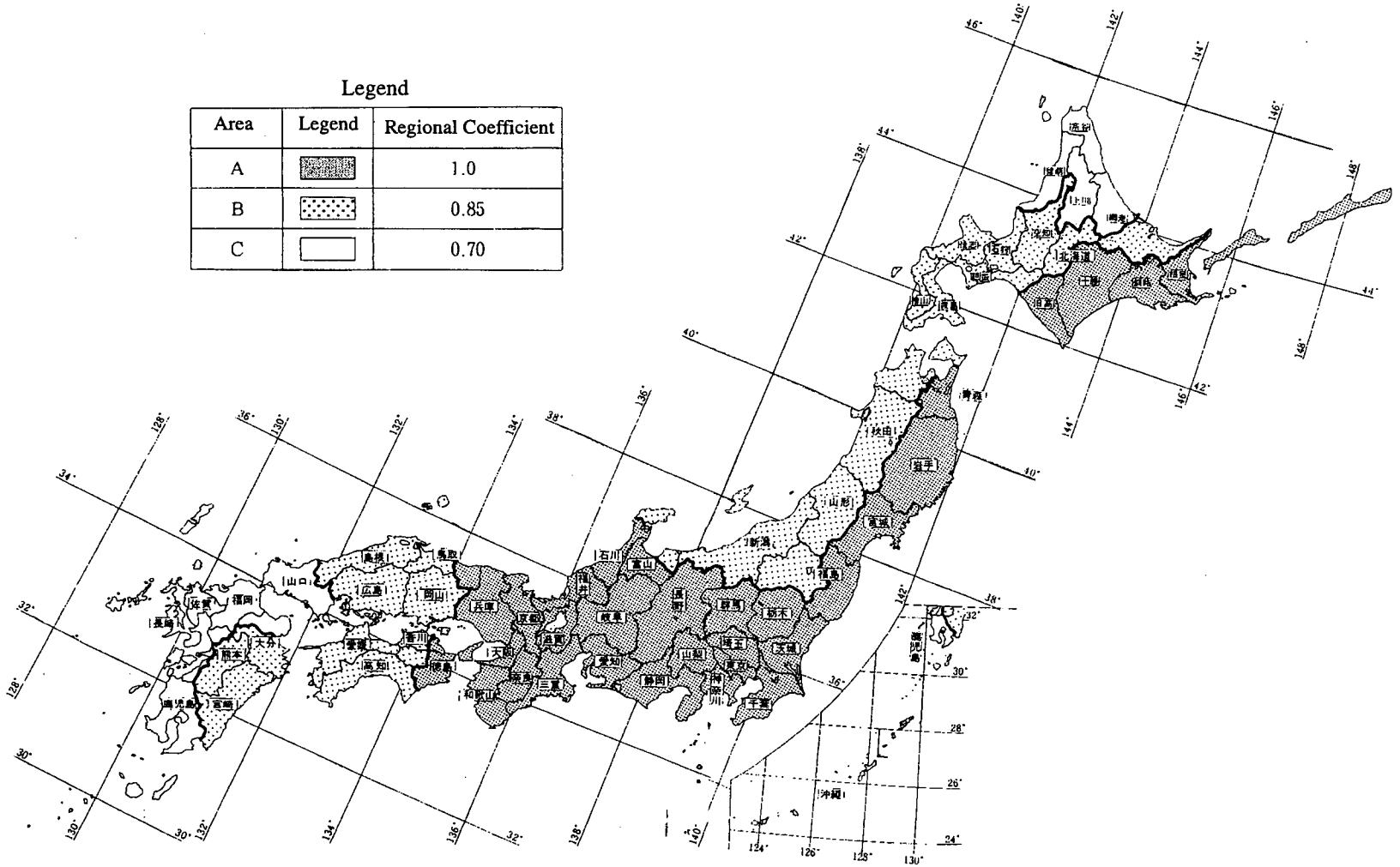


Standard Rainfall Intensity Map

Appendix 5

Legend

Area	Legend	Regional Coefficient
A		1.0
B		0.85
C		0.70



Regional Revision Coefficient c_z for Calculation of Design Seismic Coefficient

List of Reference on Highway Earthworks written in English

Name of Reference	Publisher/Editor	Year of Issue	source
Highway Earthworks Series: Manual for Slope Protection	Japan Road Association	1984	
General Guideline for Soft Ground Countermeasures	JICA	1999	Japan-Thailand Joint Study Project on Soft Clay Foundation between DOH & JICA
Manual for Design and Construction of Cement Column Method	JICA	1999	Japan-Thailand Joint Study Project on Soft Clay Foundation between DOH & JICA
Manual for Design and Construction of PVD Method	JICA	1999	Japan-Thailand Joint Study Project on Soft Clay Foundation between DOH & JICA
The Study on Road Disaster Prevention Plan in the Kingdom of Thailand: Final Report	JICA	1995	
Report of Workshop; The Study on Road Disaster Prevention Plan in the Kingdom of Thailand	JICA	1995	
Soil Erosion during and after Roadway Construction	PIARC TC12	1991	PIARC TC12: Earth works, Drainage, Subgrade: Marralesh Congress, 1991, Report R-2
LANDSLIDES Techniques for Evaluating Hazard	PIARC TC12	1997	
Equipment and Methods Specific to Soil Treatment Works	PIARC TC12	1997	
Practical Guidelines for the Design and Construction of Road	PIARC TC12	1999	
Methods and Requirements for Earthworks in order to Reduce Environmental Impact of Road Projects	PIARC TC12	2000	
Contribution to Risk Management of Existing Slopes	PIARC TC12	2000	
Natural Disaster Reduction for Roads: FINAL REPORT	PIARC	1999	PIARC Working Group on Natural Disaster Reduction (G2)
Manual for Zonation on Areas Susceptible to Rain-induced Slope Failure	Japan Geotechnical Society	1997	Asian Technical Committee on Geotechnology for Natural Hazard in ISSMFE
Standard of Japanese Geotechnical Society for Laboratory Shear Test	Japan Geotechnical Society	2000	
Remedial Measures against Soil Liquefaction from Investigation and Design to Implementation	Japan Geotechnical Society	1998	
Translation of Japanese Highway Design Manual Part 1: Earthworks	Australian Road Research Board	1985	Australian Road Research Board Resesearch Report No.51
The Advanced Cource of Practical Construction Management (for Civil Engineer)	JICA		Materials of JICA Group Training Course
Construction Management (Civil Works)	JICA		Materials of JICA Group Training Course

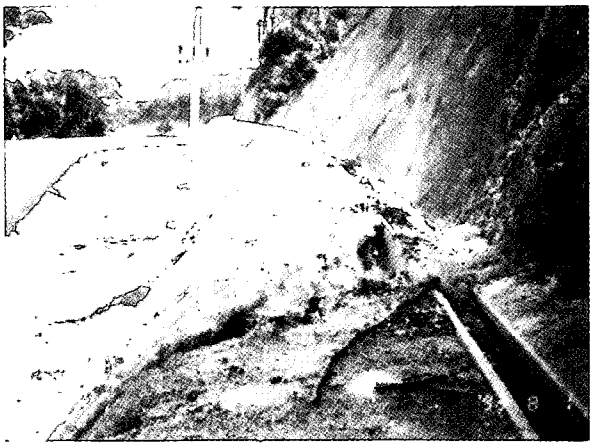


Photo 1

・Drainage ditch plugged by soil where surface erosion has advanced on the cut slope in granitic material (Thailand)

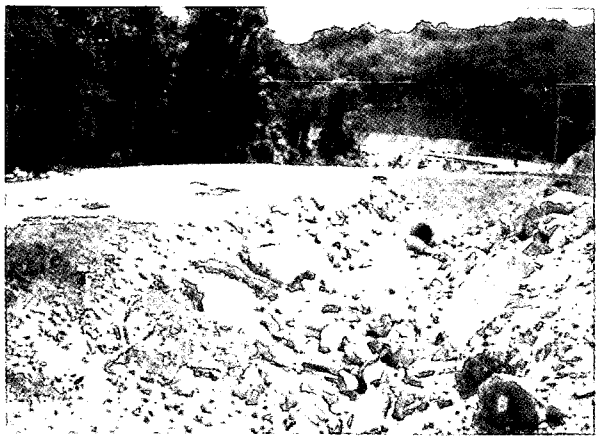


Photo 2

・ Surface drainage in embankment and cut slope section. Because the catchment area is underestimated and terminal treatment such as terracing work is inappropriate, drained water often erodes the road body or the natural surface as it flows. (Thailand)



Photo 3

・ Intensive rainfall causes slopes to collapse in the hinterlands, plugging pipe culverts with sediment and woody debris. This photo shows conditions where it is not restored with water continuing to overflow. (Thailand)

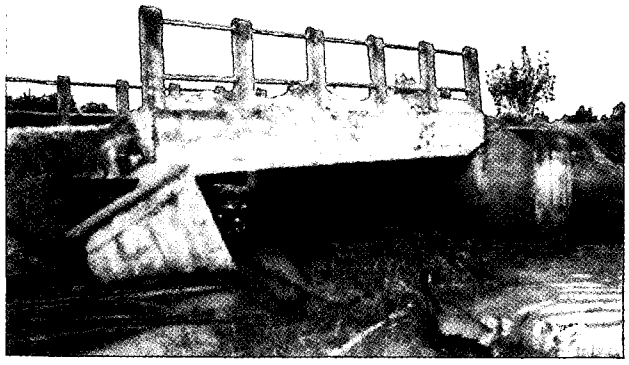


Photo 4

• Case where a bridge abutment has slipped at the crossing section of a channel on flat land. A study of works to guarantee the flow cross-section and to protect the banking section around the abutment should be done. (Thailand)

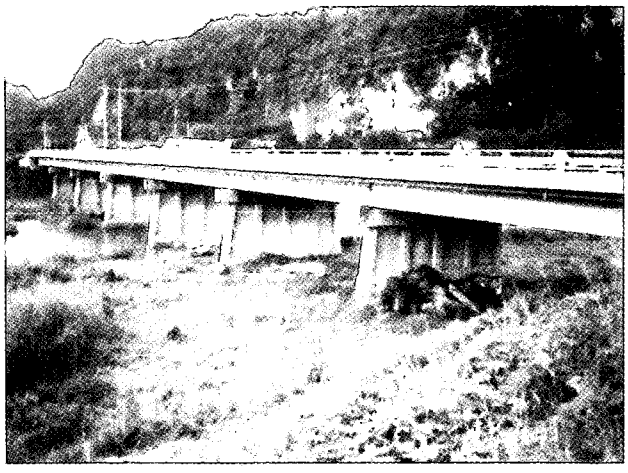


Photo 5

• At bridges at river crossings on flat land, there are many cases of slab bridges with a span of 10m or less that have inadequate clearance under the girders, or with bed scouring at the foundations of their piers. Studies to guarantee the flow cross-section or of adequate arrangement of bridge piers should be done. (Thailand)

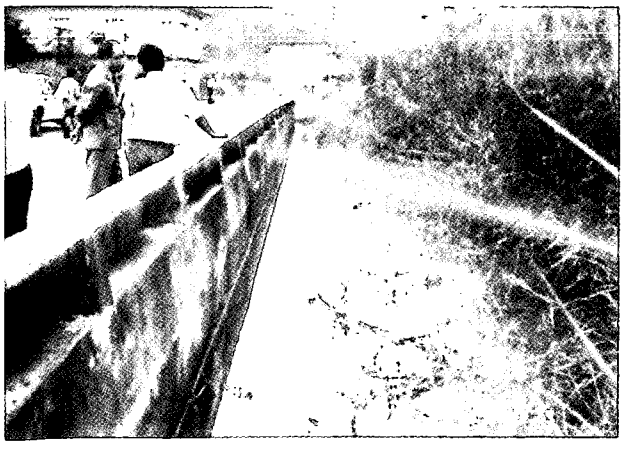


Photo 6

• Example of a bridge at a river crossing on flat land where the river has been blocked by driftwood and sediment caught on the bridge's piers. (Thailand)

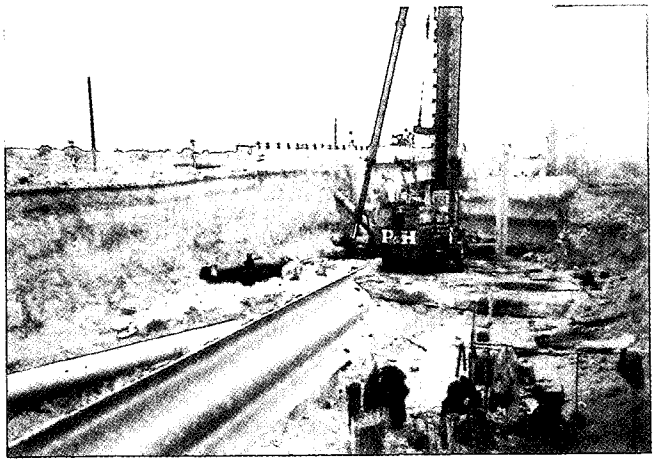


Photo 7

• Execution of the approach embankment of a bridge on a soft ground. Bridge abutment foundations often tilt, faultings are often caused by uneven settlement, and so on. Important points are countermeasures for soft ground around structures and measures in embankment (Indonesia)



Photo 8

• Case where inundations by parallel rivers have submerged and scoured the road body. The subgrade soil is very expansive and the traffic load has caused unevenness of the road surface for more than 10 kilometers. Important points are to construct revetments on water colliding front, protect the road body from water, and stabilize unusual soils (Indonesia)

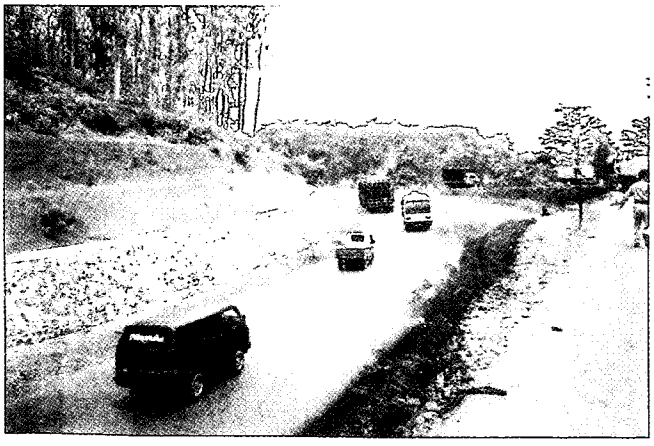


Photo 9

• Inter-city roads in developing countries must be widened and their alignments improved, so growing demand for cutting work is predicted. Important points are judging stable gradients and protecting slopes with indigenous plants (Indonesia)

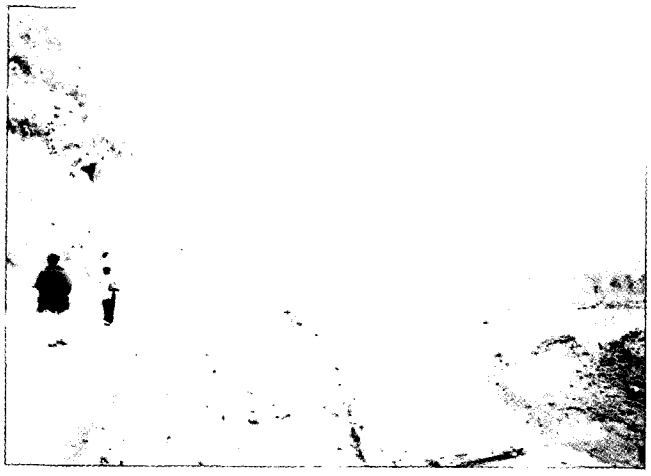


Photo 10

• Example of a large-scale slope failure caused by intensive rainfall. The collapsed soil has blocked the river, greatly transforming the river channel. (Philippines)

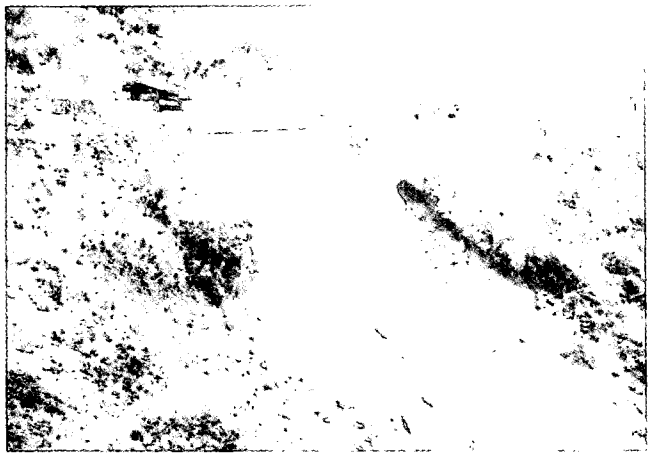


Photo 11

• A masonry retaining wall constructed on a steep slope has collapsed as a result of the slip failure of the slope that is its foundation. (Philippines)



Photo 12

• A road embankment along a river has seriously collapsed because the rise of the river water level has scoured the foundation of the revetment. Roads in mountains often run through valleys beside rivers, so it is important to carry out integrated improvement of the road embankment and the river revetment; not just the road body. (Philippines)



Photo 13

• A debris flow has blocked the upstream end of a culvert so the water flowed over the road surface, causing the failure of the road embankment on the downstream side. It is important to conduct restoration works integrated with a Sabo dam. (Philippines)

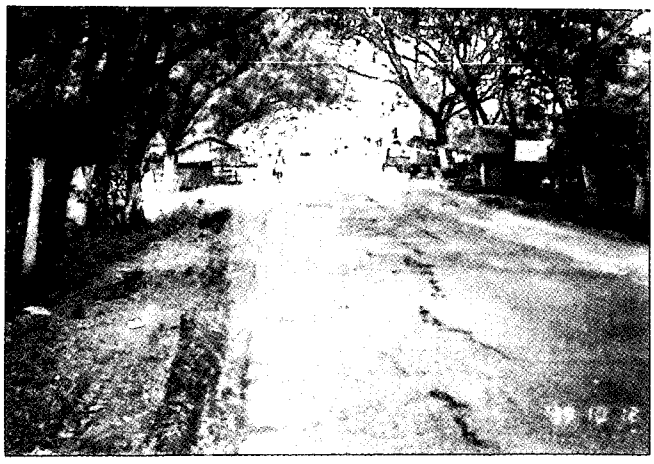


Photo 14

• Example of a case where, because the bearing strength of the subgrade soil is poor, the paving surface has subsided, creating level differences and cracking etc. (Philippines)

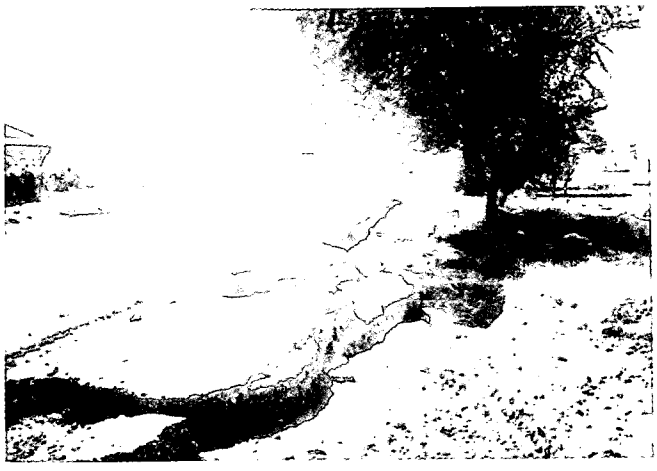


Photo 15

• Road shoulders are scoured at various locations by intensive rainfall because of unpaved road shoulders and a lack of road surface drainage or side ditches. (Philippines)

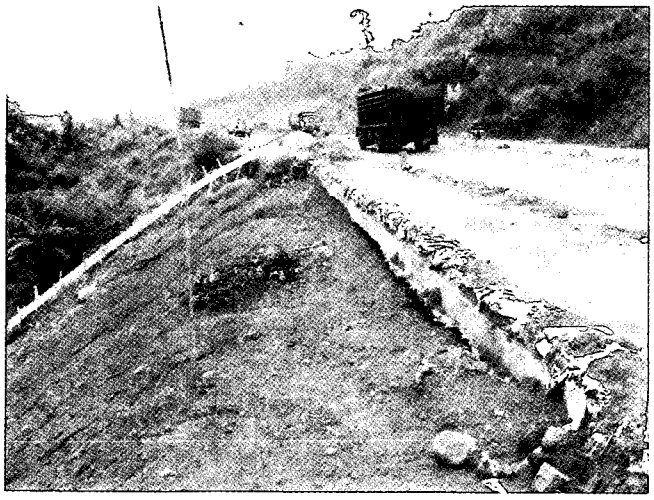


Photo 16

• Case where, because of a lack of drainage systems on natural slopes, the surfaces of the slope of a road embankment on the valley side have collapsed due to the running surface water from the mountain side. (Indonesia)