(Slope Stabilization Technology Internationalization Committee: Research and Investigation Report - Part 1)

Slope Protection Engineering Design Technical Manual

—Issues and Actions to Take for the Internationalization of Japan's Slope Protection Engineering Design Technology—



Slope Protection Technology Internationalization Committee
in The Japan Landslide Society

Foreword

Slope stabilization techniques in Japan have developed independently while incorporating overseas technologies since the Meiji era, when Dutch engineer Johannis de Rijke and Austrian researcher Amerigo Hofmann were active. Particularly during the Showa era, numerous new slope stabilization techniques were introduced, leading to the emergence of many uniquely Japanese design methods. In other words, even for similar slope stabilization techniques, there are many differences in design methods between Japan and other countries. The Subcommittee for the Internationalization of Slope Disaster Prevention Technology, established within the Research and Investigation Department of the Japan Landslide Society in 2015, became the Society's direct Committee for the Internationalization of Slope Disaster Prevention Technology in 2018. Renamed the Committee for the Internationalization of Slope Protection Technology in 2023, it has continued research activities addressing these differences in design methods between Japan and other countries.

As a result of this research, we produced two publications: the *Slope Stabilization Technology Internationalization Committee Research and Investigation Report—Part 2: A Guide to Slope Stabilization Design Techniques for Young Engineers—Historical Background and Theoretical Explanation of Issues in Overseas Expansion* (Japanese Edition), which delves into the historical background and theoretical significance of each topic; and the *Slope Stabilization Technology Internationalization Committee Research and Investigation Report—Part 1: Slope Protection Engineering Design Technical Manual— Issues and Actions to Take for the Internationalization of Japan's Slope Protection Engineering Design Technology* (Japanese and English Editions).

<Purpose of This Committee's Activities>

In recent years, there has been an increase in research and projects related to slope disasters being conducted in developing and middle-income countries overseas. The primary technical issues encountered in these contexts are as follows. The main purpose of the activities is to develop technical manuals and other resources concerning the design of slope protection works that address these issues, specifically for Japanese engineers working overseas.

(1) In regions with geological or meteorological conditions differing from Japan's, Japanese standards may not be directly applicable (e.g., cut slopes). Even in regions where they can be applied, the rationale for the standard values must be explained.

(2) Japanese engineers utilize numerical values and design methodologies published in technical standards in Japan, but since many of these reference values have been in use for a long time, few engineers understand the background of how each standard was developed.

(3) Some domestic technical standards related to slope disasters in Japan include methods that have been used for many years as customary practice but lack theoretical justification or suffer from a lack of mechanical rigor. Rather than simply applying such logically problematic methods overseas, it is necessary to provide explanations that include their background, issues, and advantages as well.

(4) While the design methods for many slope stabilization works in Japan have remained largely unchanged since the Showa era, the design approaches for the same types of works differ from those in Europe and America. The types of stability analysis equations commonly used in slope stabilization design and their application methods also differ between Japan and Europe/America. Engineers in developing and middle-income countries receive technical support from Europe and America as well as Japan. Since they use European and American

design software for slope stabilization works, they understand those design methods. In Japan, including those engaged in overseas work, many engineers are unfamiliar with Western design methods.

＜Duration of This Committee's Activities＞

April 2018–March 2026 (Preliminary Research Period: April 2015–March 2018)

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March 31, 2026

Slope Stabilization Technology Internationalization Committee

Common Provisions for the Entire Chapter

<Japanese Organization Names and Abbreviations>

ACTEC: Advanced Construction Technology Center
AIST: National Institute of Advanced Industrial Science and Technology

Forestry Agency: Forestry Agency, Ministry of Agriculture, Forestry and Fisheries
FFS: Free Frame Society

GSJ: Geological Survey of Japan

HCA: Hokuriku Construction Association

JAA: Japan Anchor Association
JASDiM: Japan Association for Slope Disaster Management
JCEA: Japan Construction Engineers' Association
JCI: Japan Concrete Institute
JEPOC: Electric Power Civil Engineering Association
JFCA: Japan Forest Conservation Association
JFECF: Japan Forest Engineering Consultants Foundation
JGS: The Japanese Geotechnical Society
JH: Japan Highway Public Corporation
JLS: Japan Landslide Society
JMA: Japan Meteorological Agency
JNR: Japanese National Railways
JPTA: Japan Pneumatic Technology Association
JRA: Japan Road Association
JRCC: Japan Railway Construction Public Corporation
JSA: Japan SABO Association
JSCE: Japan Society of Civil Engineers
JSMC: Japan Steel Materials Club
JSPA: Japan Slope Protection Association
JSSMFE: The Japanese Society of Soil Mechanics and Foundation Engineering (The society that preceded the JGS)
JTA: Japan Tunnelling Association

KSC: Kawasaki Steel Corporation
KNBI: Kita-Nihon Block Industry

MAFF:	Ministry of Agriculture, Forestry and Fisheries
MLIT:	Ministry of Land, Infrastructure, Transport and Tourism
MOC:	Ministry of Construction (Organizations that preceded MLITT)
NEXCO:	Nippon Expressway Company (NEXCO is divided into three companies: East Nippon Expressway, Central Nippon Expressway, and West Nippon Expressway)
NILIM:	National Institute for Land and Infrastructure Management
PWRC:	Public Works Research Center
PWRI:	Public Works Research Institute
RTRI:	Railway Technical Research Institute
TACPL:	Technical Association of Corrugated Pipes and Liner Plates

<Japanese Technical Guidelines, etc. referenced in this paper >

General

JSCE (1976): *Standard Specifications for Concrete Structures -Design Edition*, Japan Society of Civil Engineers (JSCE).

MLIT (2011): *Common Specifications for Civil Engineering Works*, Ministry of Land, Infrastructure, Transport and Tourism (MLIT).

Erosion Control Related

MOC (1958): *Ministry of Construction (MOC), Technical Criteria for River and Erosion Control Works (Draft)* , Sankaido Publishing.

MOC (1976): *Ministry of Construction (MOC), Technical Criteria for River and Erosion Control Works (Draft): Investigation Volume and Planning Volume*, Sankaido Publishing.

MOC (1985): *Ministry of Construction (MOC), Technical Criteria for River and Erosion Control Works (Draft): Design Volume 2*, Sankaido Publishing.

MOC (1997): *Ministry of Construction (MOC), Technical Criteria for River and Erosion Control Works (Draft): Planning Volume*, Sankaido Publishing.

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PWRI (2008): *Technical Guidelines for Slow-moving Landslide Prevention and Commentary*, Public Works Research Institute (PWRI), 142p..

JSA (2019): *New Design and Examples of Slope Failure Prevention Works: Technical Guidelines for Steep Slope Failure Prevention Works*, Japan SABO Association (JSA).

JCEA (2025): *Disaster Recovery Handbook*, Japan Construction Engineers' Association (JCEA)

Road-related

JRA (1956): *Guidelines for Highway Earthwork*, Japan Road Association (JRA)

JH (1964): *Design Guidelines for Expressways*, Japan Highway Public Corporation (JH)

JRA (1967): *Guidelines for Highway Earthwork*, Japan Road Association (JRA)

JRA (1972): *Guidelines for Slope Protection and Slope Stability Engineering*, Japan Road Association (JRA)

JRA (1986): *Guidelines for Highway Earthwork: Soil Investigation*, Japan Road Association (JRA)

JRA (1999): *Guidelines for Highway Earthwork: Temporary Structures*, Japan Road Association (JRA)

JRA (2009): *Outline for Highway Earthwork*, Japan Road Association (JRA), 416p.

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

JRA (2012): *Guidelines for Highway Earthwork: Retaining Wall*, Japan Road Association (JRA).

JRA (1980): *Specifications for Highway Bridges, Part IV: Substructures and Commentary*, Japan Road Association (JRA), Mruzen Publishing.

JRA (1994): *Specifications for Highway Bridges, Part IV: Substructures and Commentary*, Japan Road Association (JRA), Mruzen Publishing.

JRA (2012): *Specifications for Highway Bridges, Part IV: Substructures and Commentary*, Japan Road

Association (JRA), Mruzen Publishing.

JRA (2017): *Specifications for Highway Bridges, Part IV: Substructures and Commentary*, Japan Road Association (JRA) , Mruzen Publishing.

MLIT (2020): *Design Guidelines: Highways*, Ministry of Land, Infrastructure, Transport and Tourism (MLIT), Hokuriku Regional Development Bureau.

Agricultural Land Related

MAFF (2013): *Guidelines for Maintaining the Functionality of Slow-moving Landslide Prevention Facilities - Mitigation Measures Edition*, Ministry of Agriculture, Forestry and Fisheries (MAFF).

Forest Conservation Related

JFCA (1987): *Technical Standards and Commentary for Forest Conservation: Slow-moving Landslide Prevention Edition*, Japan Forest Conservation Association (JFCA).

JFCA (2013): *Technical Standards and Commentary for Forest Conservation: Slow-moving Landslide Prevention Edition*, Japan Forest Conservation Association (JFCA), 204p.

Large-diameter Drainage Well Construction Related

TACPL (2000): *Liner Plate Design and Construction Manual*, Technical Association of Corrugated Pipes and Liner Plates (TACPL)

TACPL (2009): *Liner Plate Design and Construction Manual*, Technical Association of Corrugated Pipes and Liner Plates (TACPL)

HCA (210): *Reinforced Concrete Large-diameter Design and Construction Manual*, Hokuriku Construction Association (HCA), 71p.

JEPOC (1986): *Design and Construction of Underground Structures for Electric Power Facilities*, Electric Power Civil Engineering Association (JEPOC), Sankaido Publishin, pp.573-593.

JTA (19782): *Guidelines for Investigation, Design, Construction, and Measurement of Deep Shafts and Tunnels for Underground Power Transmission*, Japan Tunnelling Association (JTA), pp.65-123.

ACTEC (1994): *Guidelines for Design and Construction of Deep Earth Retaining Structures (Draft)*, Advanced Construction Technology Center (ACTEC), pp.33-42.

Ground Anchor Construction Related

JSSMFE (1976): *Earth Anchors*, The Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE), 236p.

JSSMFE (1990): *Design and Construction Standards for Ground Anchors, and Explanatory Notes*, The Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE), 148p.

JGS (2000): *Design and Construction Standards for Ground Anchors, and Explanatory Notes*, The Japanese Geotechnical Society (JGS), 219P.

JGS (2012): *Design and Construction Standards for Ground Anchors, and Explanatory Notes*, The Japanese Geotechnical Society (JGS), 211P.

JAA (2013): *Ground anchor design/ construction instruction manual*, Japan Anchor Association (JAA), 263P.

PWRC (2004): *Ground Anchor Reaction Structure Design and Testing Manual*, Public Works Research Center (PWRC).

Concrete Crib Works Related

FFS (1983): *Guidelines for Design and Construction of Free Frame Method*, Free Frame Society (FFS), 112p.

FFS (2003): *Revised Edition: Guidelines for Design and Construction of Free Frame Method*, Free Frame Society (FFS), 111p.

JSPA (2006): *Design and Construction Guidelines for Concrete Crib Works for slope*, Japan Slope Protection Association (JSPA), 85p.

JSPA (2013): *Design and Construction Guidelines for Concrete Crib Works for slope*, Japan Slope Protection Association (JSPA), 99p.

Slope Reinforcement Method Related

JSSMFE (1986): *Reinforced Soil Engineering*, The Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE).

JGS (2011): *Manual for Design and Construction of Slope Reinforcement Method*, The Japanese Geotechnical Society (JGS).

NEXCO (2007): *Design and Construction Guidelines for Cut Slope Reinforcement Method*, Nippon Expressway Company (NEXCO).

NEXCO (2024): *Design and Construction Guidelines for Cut Slope Reinforcement Method*, Nippon Expressway Company (NEXCO).

JRCC (2001): *Guide for Design and Construction of Reinforced Soil Retaining Walls*, Japan Railway Construction Public Corporation (JRCC).

JNR (1968): *Guidelines for Design and Construction of Earth Structures (Draft)*, Japanese National Railways (JNR), Construction Bureau.

RTRI (1992): *Design Standards for Railway Structures and Commentary: Earth Structures*, Railway Technical Research Institute (RTRI).

RTRI (2007): *Design Standards for Railway Structures and Commentary: Earth Structures*, Railway Technical Research Institute (RTRI).

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1. Overview

1. 1 Purpose of This Book and Intended Readers

(1) Background

Japan has numerous official technical standards related to slope stabilization. Technical standards for reactivated slow-moving landslides exist for each of the three national organizations overseeing projects, alongside standards for steep slope failures, rockfall countermeasures, and specialized standards for road slope stabilization. The earliest document is the Ministry of Construction's Technical Standards for Rivers and Erosion Control Works (MOC, 1958), issued in 1958. However, starting in the 1970s, technical standards under other jurisdictions were successively published and have undergone repeated revisions since then. Japanese engineers place high trust in these various technical standards with such a long history.

However, Japan's technical standards manuals include several design methods that lack mechanical rigor. For example, both mechanically rigorous and mechanically non-rigorous methods for calculating the reduction in sliding force of ground anchor works are documented in official technical standards manuals. Yet, many manuals feature the mechanically non-rigorous method, making it the mainstream approach in Japan. Some technical standards explicitly state that the Fellenius formula, designed for circular slide, could be applied to non-circular slide for reactivated slow-moving type. This approach also has mechanical issues. Engineers often use the methods simply because they are listed in the technical standards in Japan, without recognizing those problems.

Moreover, few engineers question the various standard values listed in technical manuals. Even fewer engineers understand the background behind how those standard values were established. For example, in Japan's countermeasures for reactivated slow-moving landslides, the method of setting the cohesion c of the sliding surface clay based on the sliding surface depth is frequently used. However, few engineers understand the mechanical basis or the issues associated with this method. Japanese engineers unaware of this mechanical basis and these issues have recently applied this method in the field of slope failure countermeasures. There are cases where it has been directly reproduced in design manuals for overseas slope failure countermeasures developed under Japanese ODA projects. Japan's standard cut slope gradient values are also unique to Japan, derived from its specific meteorological and geological conditions. The rock mass classifications used here, “Kougan” (hard rock) and “Nangan” (soft rock), are literally translated as “Hard rock” and “Soft rock,” but they represent a distinct Japanese classification system with meanings different from the English terms “Hard rock” and “Soft rock”. Few engineers understand the definitions of “Kougan” and “Nangan”. Therefore, tables of standard cut slopes from Japan cannot be directly applied overseas. To evaluate whether design methods and standard values published in various Japanese technical standards are applicable to overseas projects, it is crucial to understand the mechanical issues inherent in Japanese design methods and standard values, along with the background and process of their establishment.

Natural Resources Canada (2012) published the “*Review of existing landslide guidelines—National technical guidelines and best practices on landslides*,” but the existing “Landslide guideline document” listed therein does not include Japan's various standard documents. This reflects the reality that the existence of various Japanese standard documents concerning slow-moving landslide and steep slope countermeasures is not recognized overseas. The reason is presumed to be the lack of prepared and publicly available English versions of Japan's various standard documents. Japanese engineers recognize Japan as an advanced nation in slope disaster prevention technology, but as this case shows, the international perception differs.

As shown in Table 1.1.2, Natural Resources Canada (2012) analyzed when each guideline is applicable. It found that most guidelines are “More applicable” for “When slope failure is a concern,” while only two of the 35 guidelines (from the USA (No.27) and Hong Kong (No.13)) are “More applicable” for “After a landslide has occurred.” The remaining 33 guidelines address preventable failures such as collapses and rockfalls, corresponding to Japan's technical standards for steep slope countermeasures.

This indicates that overseas guidelines primarily targeting reactivated slow-moving landslides—where countermeasures are implemented after sliding has been confirmed—are scarce. While it is known in Japan that the term “Landslide” overseas refers to the entire phenomenon of soil and rock movement on slopes, it is not sufficiently understood that the targets of overseas Landslide countermeasures (primarily slope failures and rockfalls) differ from those of Japanese Landslide countermeasures (reactivated slow-moving landslides, slope failures, rockfalls, rock mass collapses, debris flows, etc.).

Table 1.1.1 List of Landslide Guidelines by Country(Created using data from Natural Resources Canada (2012))

No.	Country/Region	Primary Topics	Year of Publication
1	Australia	Landslide zoning	2007
2	Australia	Landslide risk management	2000, 2007
3	Australia	General guide to understanding landslides	2007
4	Canada	Risk management	1997
5	Canada	Risk management principles	2010
6	Canada/BC	Management of landslide-prone terrain	1994
7	Canada/BC	Landslide risk case studies in the forest sector	2004
8	Canada/BC	Management of landslides in the forest sector	2008
9	Canada/BC	Landslide assessments in the forest sector	2009
10	Canada/BC	Landslide assessments for residential development	2010
11	Canada/Québec	Use of landslide maps for loss reduction	2005
12	China/Hong Kong	Risk criteria for landslides	1998
13	China/Hong Kong	Geotechnical manual for slopes	2000
14	China/Hong Kong	Landslide debris-resisting barrier design	2000
15	China/Hong Kong	Guide to engineered slope maintenance	2003
16	China/Hong Kong	Natural terrain landslides	2003
17	EU	Managing landslides in urban areas	2000
18	Germany	Disaster risk management	2004
19	India	Management of landslides and snow avalanches	2009
20	International	Landslide zoning	2008
21	International	Risk management principles	2009
22	International	Open pit slope design	2009
23	Malaysia	Slope maintenance in Malaysia	2006
24	New Zealand	Planning policy and approval requirements	2007
25	Switzerland	Landslide mapping and land use planning	1997
26	UK/Scotland	Landslides in peat: hazard and risk assessments	2006
27	USA	Landslide investigation and mitigation	1996
28	USA	Landslides and planning	2005
29	USA	General guide to understanding landslides	2008
30	USA/California	Static and seismic slope stability assessments	2002
31	USA/California	Landslide management and storm damage response	2003
32	USA/Colorado	Landslide guide for state/local government planning	1989
33	USA/Missouri	Engineering policy guidelines for slope design	2011
34	USA/Utah	Evaluation of landslides	1996
35	USA/Utah	Debris flow hazards on alluvial fans	2005

※Source: Natural Resources Canada(2012): *Review of existing landslide guidelines －National technical guidelines and best practices on landslides*

Table 1.1.2 Analysis Results of Landslide Guidelines Worldwide(Created using data from Natural Resources Canada (2012))

No.	Country/Region	Year of Publication	Who				What						Where				When				
			Engineers/Geoscientists/Professionals	General public/non-technical audience	Land use planners	Regulators	Responsibilities	Land use planning	Zoning	Risk assessment/management	Site investigation/failure mechanisms	Mitigation/remediation	Proposed development areas	Developed areas	Potentially unstable slopes	Landslides occurred	All stages of land use planning	When slope failure is a concern	When risk management is needed	After a landslide has occurred	When mitigation/remediation is needed
1	Australia	2007	●	×	●	●	●	●	×	●	○	○	●	●	●	○	●	●	●	×	●
2	Australia	2000, 2007	●	×	●	●	●	●	●	○	×	○	●	●	○	○	●	●	●	×	○
3	Australia	2007	×	●	○	○	×	○	×	×	×	○	○	○	○	○	×	●	×	○	○
4	Canada	1997	●	×	○	○	●	○	×	●	×	×	○	○	○	×	○	●	●	×	×
5	Canada	2010	●	×	○	○	●	○	×	●	×	×	○	○	○	×	○	●	●	×	×
6	Canada/BC	1994	○	○	○	○	×	○	×	×	○	○	○	×	○	×	○	●	×	×	○
7	Canada/BC	2004	●	×	○	×	×	○	×	●	○	○	●	○	●	×	×	●	●	○	×
8	Canada/BC	2008	●	×	×	●	●	×	×	×	×	○	○	×	○	×	○	●	○	×	○
9	Canada/BC	2009	●	×	×	●	●	×	×	×	○	○	○	×	○	×	×	●	×	×	○
10	Canada/BC	2010	●	×	○	●	●	●	×	×	●	○	●	×	●	×	●	●	○	×	○
11	Canada/Québec	2005	○	×	●	○	×	●	●	○	×	×	●	●	●	○	●	●	●	○	○
12	China/Hong Kong	1998	●	×	○	○	×	○	×	●	○	○	○	○	○	×	●	●	●	×	●
13	China/Hong Kong	2000	●	×	×	×	×	×	×	×	●	●	○	○	●	●	○	●	×	●	●
14	China/Hong Kong	2000	●	×	×	×	×	○	×	×	○	●	○	○	●	○	○	●	×	×	●
15	China/Hong Kong	2003	○	×	×	×	○	×	×	×	○	○	×	●	●	×	○	●	×	×	×
16	China/Hong Kong	2003	●	×	○	○	○	●	○	○	●	●	○	○	○	○	○	●	○	○	●
17	EU	2000	○	○	○	○	○	○	○	×	○	○	○	○	●	○	○	●	×	○	●
18	Germany	2004	×	×	×	○	○	×	×	●	×	○	×	×	×	×	○	○	●	×	○
19	India	2009	○	×	×	●	●	×	×	○	○	○	×	○	○	×	×	●	○	×	○
20	International	2008	●	×	●	●	●	●	●	○	×	○	●	●	○	○	●	●	●	×	○
21	International	2009	●	×	○	○	●	○	×	●	×	×	○	○	○	×	○	●	●	×	×
22	International	2009	●	×	×	×	×	×	×	●	●	●	○	×	●	○	○	●	×	○	●
23	Malaysia	2006	○	×	×	×	○	×	×	×	○	○	×	●	●	×	○	●	×	×	×
24	New Zealand	2007	○	●	●	●	×	●	×	○	×	○	●	×	○	×	●	●	○	×	○
25	Switzerland	1997	●	○	○	○	×	○	●	○	●	○	○	●	○	○	○	●	○	○	×
26	UK/Scotland	2006	●	×	○	○	×	○	×	○	●	●	○	○	○	○	○	●	○	○	○
27	USA	1996	●	×	×	×	×	×	×	○	●	●	○	○	●	●	○	●	○	●	●
28	USA	2005	×	○	×	○	×	●	×	○	×	○	●	×	○	×	●	●	○	×	●
29	USA	2008	×	●	○	○	×	○	×	×	×	×	○	○	○	×	×	●	×	○	×
30	USA/California	2002	●	×	○	○	○	○	○	×	●	●	○	○	●	○	○	●	×	○	●
31	USA/California	2003	○	×	×	○	○	×	×	×	○	●	×	●	○	○	×	●	×	○	●
32	USA/Colorado	1989	×	○	○	●	×	○	○	×	○	●	○	○	○	○	○	●	○	○	●
33	USA/Missouri	2011	●	×	×	○	×	×	×	×	●	●	●	○	●	○	●	●	×	×	●
34	USA/Utah	1996	●	×	×	×	×	×	×	×	●	○	○	○	○	○	×	●	×	○	○
35	USA/Utah	2005	●	×	●	○	×	×	○	○	○	●	○	×	×	○	○	○	○	×	●

Legend: ●More applicable；○Applicable;；×Less applicable or not applicable

※Source: Natural Resources Canada(2012): *Review of existing landslide guidelines －National technical guidelines and best practices on landslides*

Additionally, there are issues concerning differences between commercially available design software overseas and in Japan. Since the Showa era, Japan has continuously specified stability analysis equations, methods for setting soil strength parameters, and key design parameters for slope protection works in official technical standards, maintaining strict specification requirements to this day. Japanese commercial design software prioritizes compliance with these various technical standards and therefore does not support stability analysis equations or design methods differing from those described in the standards. Conversely, Western design software widely adopted in developing countries generally supports probabilistic evaluation of soil strength parameters and performance-based design for slope protection works. It also accommodates limit state design methods compliant with Eurocode 7. Furthermore, the stability analysis equations and slope protection design methods employed vary between different design software packages. The types of Western design software used in developing countries vary by nation. Japanese engineers must recognize that engineers in developing countries routinely use Western design software and understand the differences between Japanese and Western design software.

Because these various issues arising when deploying Japanese slope protection technology overseas are not widely recognized in Japan, this book has been compiled as a guidebook for slope protection design in overseas projects.

(2) Purpose of This Book

This book is not an introductory guide to designing various slope protection structures nor a general technical manual. It is a specialized reference focusing on key issues encountered when implementing technical transfer related to slope protection design overseas, primarily explaining actions to take.

The objectives of this book are as follows:

1) To serve as technical reference material for Japanese engineers involved in slope disaster prevention projects overseas when instructing engineers in the target country.
2) To serve as technical reference material for overseas counterparts to understand the fundamental concepts and critical aspects of Japanese slope protection design techniques.
3) To function as substitute technical reference material in developing countries where specialized technical literature on slope protection design is scarce.
4) To serve as technical reference material when developing slope protection design manuals and similar documents in developing countries.

This book is intended for engineers possessing knowledge and experience in the design of slope stabilization works. If the reader lacks general knowledge regarding the design of the relevant slope stabilization works, it is a prerequisite for using this book that they acquire the necessary knowledge by consulting technical literature and technical standards related to those works.

(3) Target Readers

1) Japanese engineers active in ODA projects and similar initiatives

This manual is intended for use by Japanese engineers who have design experience with the targeted countermeasures within Japan and are capable of instructing young engineers in design techniques, when transferring technology to their counterparts in overseas countries.

It is difficult for engineers without experience in design work or technical instruction in Japan to transfer or instruct design techniques overseas. Japanese engineers without such experience in the targeted countermeasures who use this document overseas must thoroughly study relevant technical literature and various technical standards in Japan, or receive on-site instruction from experienced engineers, to reach a level where they can instruct design techniques within Japan.

2) Counterpart for ODA Projects

This document serves as technical reference material for counterparts when Japanese engineers conduct technology transfer activities such as design technology overseas.

1. 2 Legal Background of Landslide Disaster Countermeasures in Japan and Related Public Works

Japan has a fundamental law, the Basic Act on Disaster Management (enacted in 1961), which establishes the framework for responding to all types of natural disasters. It clearly states that it is the duty of the national government, local governments, and other public institutions to protect the nation's territory and the lives, bodies, and property of its citizens from disasters. Within this Basic Act, the government aims to establish and promote comprehensive and planned disaster prevention administration by defining the fundamentals of disaster countermeasures. These include the formulation of disaster prevention plans, disaster prevention, emergency disaster response, disaster recovery, and fiscal and financial measures related to disaster prevention.

Under this basic law, specific laws detailing countermeasures for each type of disaster have been enacted. Table 1.2.1 lists the laws concerning sediment disaster countermeasures and the corresponding public works projects. Reactivated slow-moving landslides tend to concentrate in specific geological zones, occurring under limited site conditions (geology, geological structure, groundwater conditions, etc.). However, in Japan, where mountains and hills dominate the landscape, slope failures (including surface and deep-seated failures) on steep slopes occur nationwide across various geological zones during torrential rains or earthquakes. Their occurrence frequency is far greater than that of reactivated slow-moving landslides. This is confirmed by the distribution of slope failures during past torrential rains and earthquakes. Both are landslides classified by their movement patterns as slides. However, because the approaches to hard countermeasures differ significantly between the two, in Japan, the names and budgets of the respective countermeasure projects differ, and the laws serving as the basis for these projects also differ. The former is addressed through the “Slow-moving Landslide Prevention Act (enacted 1958)” with public works projects called “Slow-moving Landslide Countermeasure Projects.” The latter is addressed through the “Act on Prevention of Disasters Caused by Collapse of Steep Slopes (enacted 1969, abbreviated as the Steep Slope Act)” with public works projects called “Steep Slope Collapse Countermeasure Projects,” primarily implementing hard countermeasures.

Japan has approximately 75,000 confirmed dormant slow-moving landslides indicating past landslide activity

(Disaster Prevention Research Institute: Dormant Landslide Distribution Map Database). Slow-moving landslides targeted for countermeasure projects are limited to those confirmed to have reactivated and those deemed highly hazardous. Therefore, the targeted disaster type is described as "reactivated slow-moving landslides." Initial slow-moving landslides are difficult to prevent proactively; slow-moving landslide countermeasure projects are applied only when reactivation is confirmed in the displaced mass after an initial slow-moving landslide or when such a risk exists.

Other relevant laws include the Erosion Control Act (the SABO Act enacted 1897), which addresses measures against degradation in upstream mountainous areas and debris flows, and the New River Act (enacted 1964), which defines river classifications, management methods, flood control, and utilization. Projects implemented under the SABO Act are referred to as "erosion control projects." Furthermore, the New River Act clarifies that major rivers are managed collectively by the national government (Ministry of Land, Infrastructure, Transport and Tourism).

Soft measures, such as land use restrictions related to debris flows, reactivated slow-moving landslides, and collapses, are stipulated by the "Act on the Promotion of Measures to Prevent Sediment Disasters in Sediment Disaster Warning Areas, etc." (enacted in 2000, abbreviated as the Sediment Disaster Prevention Act).

Table 1.2.1 Laws Concerning Sediment Disaster Countermeasures in Japan and Corresponding Public Works Project Names

Disaster Type	Corresponding Law (Year Enacted)	Public Works Project Name
All Natural Disasters	the Basic Act on Disaster Management (1961)	All Disaster Countermeasures Projects
Reactivated slow-moving Landslides	the slow-moving Landslide Prevention Act (1958)	Slow-moving Landslide Countermeasures Projects (Primarily Hard Countermeasures)
Steep Slope Collapse	the Steep Slope Prevention Act (1969)	Steep Slope Collapse Countermeasure Projects (primarily hard measures)
Debris Flows	the Erosion Control Act (1897)	Erosion Control Projects (primarily hard measures)
River Flooding	the New River Act (1964)	River Projects (primarily hard measures)
Debris Flows, Reactivated slow-moving landslides, Steep Slope Collapses	the Sediment Rerated Disaster Prevention Act (2000)	No specific project name (soft measures)

In many countries receiving technical cooperation, laws related to landslide disaster countermeasures are not yet established. Furthermore, in many of these countries, it is not clearly stipulated by law that disaster prevention and other preemptive measures are the responsibility of national or local governments. This lack of legal clarity is one reason for the passive approach to landslide disaster prevention.

1. 3 Structure and Coverage of This Book

This chapter corresponds to one slope stabilization structure per chapter. The chapter structure and content are as follows.

2. Design of XX Works (One chapter per work type)

2.1 Overview of XX Works

Describe the overview of the countermeasure works. In addition to uniquely Japanese work types like large-diameter drainage wells and shotcrete crib works, works such as ground anchors (not rock anchors) and concrete crib works are also not common slope stabilization works overseas. Provide explanations to convey the content and characteristics of the construction methods.

2.2 Key Issues in Internationalization and Actions to Take

This section details the issues encountered overseas and how to address them, broken down into sub-items by challenge type. It covers not only issues actually faced abroad, but also those arising from domestic factors, such as many Japanese engineers being unable to adequately explain their work.

2.3 Design Considerations

Summarize key sections from relevant technical reference materials (e.g., key sections from the Design Guidelines for Steel Pipe Piles for slow-moving Landslides). Content should be tailored to the circumstances of target countries with limited technical literature. Since it is not possible to include all content from Japanese technical reference materials, summarize and include only particularly important sections.

2.4 Example Descriptions of Items Requiring Attention When Developing Technical Standards

This section provides example descriptions of items requiring attention when developing technical standards for target countries. It shows example descriptions for expressing the issues and actions to take explained in Section 1 of this chapter specifically as technical standards and their explanations.

2.5 Examples of Descriptions for Other Technical Standards, etc.

For items not listed as issues in Section 1 of this chapter, examples of descriptions are provided for those items where specific technical standards and explanatory notes are considered desirable.

Furthermore, differences in understanding issues regarding “Landslide” and those in stability analysis are addressed in a separate chapter, as they represent common issues for all slope stabilization works.

1. 4 Target Slope Stabilization Works

The slope stabilization works covered in this document are the following types.

表 1.4.1 **Slope Stabilization Works Covered in This Book**

No.	Type of Work	Remarks
1	Slope Stability Analysis	Not a stabilization work itself, but included as a technology related to the design of various stabilization works
2	Cut Slope Works	Primarily covers cut slope works for road cut slopes, etc.
3	Large-diameter drainage well Works	Countermeasures for reactivated slow-moving restratified landslides. Covers both steel and RC (Reinforced Concrete) types.
4	Slope Reinforcement Works	Primarily covers soil nailing. Only internationalization issues are noted.
5	Ground Anchor Works	Measures against slope failures and reactivated slow-moving landslides
6	Concrete Crib Works	Primarily covers shotcrete crib works. Only internationalization issues and design considerations are noted.

These work categories were selected as those in which adopting Japanese design methods directly in overseas projects, such as ODA projects, tends to cause particular problems. While other construction methods were initially considered, they were excluded from this book due to time constraints and manpower limitations.

However, some design methods for countermeasures not covered in this book also involve internationalization issues. For example, the design methodology for steel pipe pile works against slow-moving landslides is unique to Japan. It differs from the design theory for restraint piles adopted by Western design software. It also presents issues such as mechanical problems in the calculation method for required prevention force, differing criteria among regulatory agencies for judging the short-term and long-term strength of steel materials, and the problem of Japan's unique standard pile spacing values based on slip surface depth. Shaft pile construction also presents issues for internationalization, including mechanical issues in the calculation method for required prevention force and differing criteria among regulatory agencies for classifying piles as long or short. Clarifying these internationalization issues will be necessary in the future.

<What Is Required Stabilizing Force?>

The following is a supplementary explanation of the required stabilizing force. In Japan, restraint measures such as steel pipe piling, ground anchor, and shaft works have been widely adopted as countermeasure works for slow-moving landslides. The required stabilizing force is a concept common to these restraint measures; it refers to the value Pt that must be subtracted from the denominator, or the value Ps that must be added to the numerator of the stability analysis equation to achieve the target safety factor through the effectiveness of the restraint measures. Therefore, it differs from the horizontal force used in the design of steel pipe piling or the design anchor force in ground anchor works.

1. 5 Conducting Interviews and Gathering Information

Many of Japan's technical standards for slope stabilization/protection countermeasures were first published or significantly revised in the 1970s. The Subcommittee on Internationalization of Slope Disaster Prevention Technology, established within the Research and Survey Department of the Japan Landslide Society in 2015, conducted interviews in 2017 with experts involved in the creation of various technical standards in the 1970s and familiar with the circumstances of that time. The subjects of these interviews were the following three individuals.

Table 1.5.1 Experts Interviewed in 2017

No.	Expert	Main Background
1	Dr. Hisao Fujita	• Former Director of the Erosion Control Division, Public Works Research Institute, Ministry of Construction • Former President of the Japan Landslide Society • Doctor of Philosophy
2	Dr. Seishi Okuzono	・Former Head of the Tunnel and Slope Research Laboratory, Japan Highway Public Corporation Testing Laboratory ・Former Professor, Kyushu Sangyo University ・Doctor of Engineering
3	Dr. Hiroyuki Nakamura	• Former Head of the Landslide Research Laboratory, Public Works Research Institute, Ministry of Construction • Former Professor, Tokyo University of Agriculture and Technology • Former President, Japan Landslide Society • Doctor of Agricultural Sciences

These interview findings were shared among the members of the Subcommittee on Internationalization of Slope Disaster Prevention Technology in 2017.

2. Differences in Perception of “Landslide” Between Overseas and Japan

This chapter explains the differences in perception of “Landslide” between overseas and Japan as a common issue in the internationalization of slope protection design.

(1) Issues of Internationalization

In Japan, the Japanese term “Jisuberi” is generally translated into English as “Landslide.” Overseas, ‘Landslide’ is often used as a comprehensive term for slope movement phenomena. For classification, Varnes' classification is used worldwide, categorizing phenomena as “fall,” “topple,” “slide,” “spread,” “flow,” “creep,” and others. Varnes' classification is also standard within Japanese academic fields. However, in Japanese slope countermeasure projects, phenomena are classified as “reactivated slow-moving landslides,” “collapses,” “debris flows,” “rockfalls,” “rock mass failures,” etc., limiting the use of ‘Jisuberi’ to “re-sliding type slow slides.” Furthermore, slope movements addressed under Japan's slow-moving Landslide Prevention Act differ from those covered by the Steep Slope Act (Act on Prevention of Disasters Caused by Collapse of Steep Slopes). Existing international landslide guidelines predominantly focus on preventive measures, with extremely few emphasizing post-occurrence countermeasures (see Table 1.1.2). In other words, overseas Landslide guidelines often define “Landslide” as collapses or rockfalls where preventive measures are feasible, and frequently exclude countermeasures for reactivated slow-moving landslides.

The classification used in Japan's slope countermeasure projects emphasizes differences in the approach to countermeasures and methods of stability analysis. However, this is a classification unique to Japan, and its necessity is not understood by overseas engineers.

(2) Explanation

1) Differences in Landslide Classification Methods

Varnes' classification categorizes landslides based on the different patterns of sediment movement. Japan also adopts this classification for academic purposes. Overseas, however, Varnes' classification is also used in slope stabilization projects. Technical literature on reactivated slow-moving landslides in Japan often begins by explaining the difference between reactivated slow-moving landslides and collapses. This is to clearly distinguish the targets of slow-moving landslide countermeasure projects and steep slope collapse countermeasure projects. While both are classified as “Slides” under Varnes' classification, the approach to countermeasure projects differs between reactivated slow-moving landslides and collapses. reactivated slow-moving landslides are relatively large-scale mass movement phenomena, where the length of the moving mass can reach approximately 1 km, requiring substantial costs for countermeasures. The most critical aspect of countermeasures is understanding the distribution of slip depth. Since the movement rate is relatively slow, post-event countermeasures become the primary focus after landslide movement is confirmed. Conversely, collapses occurring on steep slopes often cause damage by the time movement is detected. Therefore, pre-event surveys identify potential collapse sites before movement occurs, and countermeasures are implemented. In other words, pre-event countermeasures become the primary focus.

The methods for stability analysis also differ between reactivated slow-moving landslides and collapses. For reactivated slow-moving landslides, the distribution of the actual failure plane, the “slip surface,” is identified

through various investigations, and stability analysis is performed on the moving mass on this actual slip surface. In contrast, for collapses, since no actual slip surface exists on the target slope, it is necessary to infer the potential depth of the slip surface from nearby collapses conditions or estimate it through repeated trial stability analyses. Similarly, for falls, the investigation methods and countermeasure approaches differ significantly between rockfalls and rock mass collapses. For flows, the approach to countermeasure projects also differs between flows on slopes (earth flows) and flows in streams (debris flows). Implementing countermeasure projects necessitates a Japanese-specific classification system, as the concepts for countermeasure projects and stability analysis differ for debris flows, rockfalls, and rock mass collapses.

2) Characteristics of Overseas Landslide Guidelines

As shown in Table 1.1.2, Natural Resources Canada (2012)'s “Review of existing landslide guidelines—National technical guidelines and best practices on landslides” analyzes when each country's landslide guidelines are applicable. Most guidelines correspond at the “More applicable” level for “When slope failure is a concern,” while only two out of 35 guidelines (USA and Hong Kong) correspond at the “More applicable” level for “After a landslide has occurred.” Furthermore, only two out of 35 guidelines (USA and Hong Kong) correspond at the “More applicable” level for “When mitigation/remediation is needed. The guideline level which is “more applicable after a landslide has occurred” was only two out of the total 35 guidelines (USA and Hong Kong). Furthermore, 13 out of the total 35 guidelines were “more applicable, when mitigation/remediation is needed." Most guidelines primarily describe preventive measures for landslides.

For large-scale reactivated slow-moving landslides, preemptive measures are rarely implemented before landslide movement is observed. In other words, the “Landslide” addressed in overseas guidelines refers to collapses, rockfalls, debris flows, and similar phenomena where preemptive measures are feasible, corresponding to Japan's technical standards for steep slope countermeasures and debris flow countermeasures. Analysis of what each country's landslide guideline addresses revealed that many guidelines corresponded to “responsibilities,” “land use planning,” “zoning,” “risk assessment,” and “site investigation/failure mechanisms.” Among all 35 guidelines, only 11 addressed “mitigation/remediation” at the “More applicable” level. Section 3.5, which details the mitigation and remediation content of each corresponding guideline, specifically describes the following countermeasures: “landslide debris-resisting barriers,” “including rock/boulder fences,” “gabions,” “reinforced concrete retaining walls,” “earth fill berms,” and “check dams.” There is no explanation regarding countermeasures effective for reactivated slow-moving landslides. Reviewing the analysis content, the 13 guidelines addressing “when mitigation/remediation is needed” appear to place greater emphasis on ‘mitigation’ rather than “remediation.”

Thus, a significant difference exists between overseas approaches—which primarily target “Landslide” phenomena like collapses, rockfalls, and debris flows, emphasizing preemptive measures—and Japan's approach. Japan clearly distinguishes ‘Jisuberi’ from these phenomena, requiring preemptive measures before movement confirmation for collapses, rockfalls, and debris flows, while focusing on post-movement confirmation countermeasures for “Jisuberi.” Many Japanese engineers remain unaware of this fundamental distinction.

(3) Response

The classifications used in Japan—such as reactivated slow-moving landslides, collapses, debris flows,

rockfalls, and rock mass failures—are designed for implementing countermeasures and are more suitable than Vernes' classification for this purpose. It is necessary to fully explain this to the responsible personnel in the target country and ensure their understanding. Furthermore, it is important to recognize the difference between Japan, which implements countermeasure projects for all slope disasters including reactivated slow-moving landslides, and overseas countries where the primary project targets are collapses, rockfalls, debris flows, etc.

References

Natural Resources Canada(2012): *Review of existing landslide guidelines —National technical guidelines and best practices on landslides*, Geological survey of Canada, Open file 7058, 13p.

3. Slope Stability Analysis

3. 1 Key Issues and Actions to Take in Internationalization

3. 1. 1 Application Practices of the Fellenius Equation to Non-Circular Slides

(1) Issues

The Fellenius formula and modified Fellenius formula are stability analysis formulas designed for circular slide. However, in Japan, there is a practice of applying them to non-circular slide. This practice lacks mechanical rigor, yet many Japanese engineers are unaware of this issue. This lack of causes is the problem.

Conversely, many English-language stability analysis software packages sold in Europe and America do not allow selection of the Fellenius formula when analyzing slides with non-circular slip surface. This reflects a lack of awareness that the Fellenius formula can be applied to non-circular slide. This further highlights the need to recognize the problems with the Japanese practice and work towards improvement.

(2) Explanation

Applying the Fellenius formula or modified Fellenius formula to non-circular slide results in applying scalar addition rules to vector forces, which violates the laws of mechanics. Furthermore, the calculated values become unknown physical quantities. For example, as shown in Figure 3.1.1, when the sliding forces of adjacent segments are *a* (=80 kN) and *b* (=20 kN), the sum according to the laws of mechanics should be approximately 92 kN. However, the Fellenius formula evaluates the sum of the sliding forces of the two segments as 100 kN. Evaluating this value 100 kN as a force violates the law of conservation of energy, making this value an unknown physical quantity that is not a force. Thus, the scalar sum of vectors does not satisfy universally accepted laws of mechanics. In Japan's slow-moving landslide countermeasure projects, this unknown quantity is used to calculate the required stabilizing force of restraint measures, creating a mechanical contradiction.

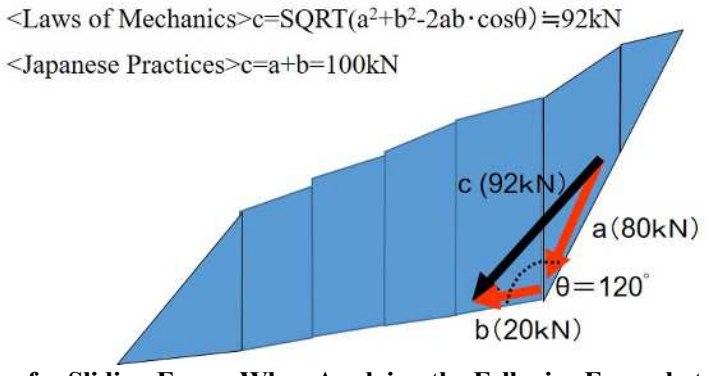


Figure 3.1.1 Addition Law for Sliding Forces When Applying the Fellenius Formula to Non-Circular Slide

Stability analysis in Japan's slow-moving landslide countermeasure projects (projects targeting reactivated slow-moving landslides) is primarily intended not to evaluate the true stability of the landslide, but to assess the change in the safety factor due to the countermeasure works as a relative change value from the current state. In practical terms, since using stability analysis formulas for non-circular slide does not result in significant differences, this practice continues. The simplicity and ease of understanding of the formula, along with the straightforward method for calculating the required stabilizing force for restraint measures, are also reasons why the Fellenius formula and modified Fellenius formula are widely used. Around 1958, when Japan's Slow-moving

Landslide Prevention Act was enacted, even scientific calculators were unavailable, and stability analysis was performed using slide rules and mechanical calculators, leading to the adoption of the Fellenius formula. In Japan, where practical experience is highly valued, this practice has continued to the present day.

However, in Japan, the simplified Janbu method has been used as standard for non-circular landslides in certain slow-moving landslide countermeasure projects since the 1980s, with over 30 years of proven results. It has been applied in countless landslide countermeasure projects during this period, and no problems have arisen in the design of countermeasure works related to stability analysis.

With advances in computer technology, the computational difficulty of the Fellenius method and the simplified Janbu method is no longer an issue. To promote landslide countermeasure projects overseas, it is considered that the simplified Janbu method, which is mechanically almost consistent, should be applied to the slides with non-circular slip surfaces.

(3) Actions to Take

The technical manual for SLOPE/W, widely used overseas, explains that the Fellenius formula, which contains many unrealistic factors, should not be used for circular slides. Given this, it is unlikely that engineers and researchers in Europe and the United States would accept applying the Fellenius formula to non-circular slides. Furthermore, the Western design software commonly used by engineers in developing countries is not configured to apply the Fellenius formula to non-circular slides. From this reason as well, this method of application should be avoided. For Japan's overseas projects, stability analysis formulas corresponding to both circular slide and non-circular slide should be applied. The simplified Janbu formula, the most convenient method for non-circular slide, has a proven track record in Japan's slow-moving landslide countermeasure projects and is highly reliable in practical applications.

3. 1. 2 Differences in Understanding Regarding Strict Limit Equilibrium Methods and Simplified Limit Equilibrium Methods

(1) Issues

Overseas, English-language stability analysis software developed in Europe and elsewhere is readily available. Consequently, exact limit equilibrium methods such as the Spencer method and M&P method are frequently used for stability calculations of embankments. In Japan's steep slope failure countermeasure projects, the Fellenius formula is widely used, while in slow-moving landslide countermeasure projects, the Fellenius formula and simplified Jambou formula are frequently employed. However, using these methods directly overseas raises questions: Why use simplified limit equilibrium methods? Why not use the Spencer method or M&P method?

(2) Explanation

1) Fundamental Issues with the Limit Equilibrium Method

The fundamental issue lies in the assumptions inherent to the limit equilibrium method:

a) The ground is assumed to be rigid.

b) Loads acting on a portion of the moving layer are assumed to be transmitted 100% to the entire moving layer.

c) It assumes that loads acting on a portion of the moving layer are not transmitted to the fixed layer.

These assumptions will be explained in detail.

The stability analysis formula of the limit equilibrium method treats the moving body as a rigid body and determines the slope's safety factor based on the structural mechanical equilibrium of forces between slices. It completely disregards the ground's elastoplastic properties. The limit equilibrium method incorporates structural mechanics techniques, applying the condition that horizontal forces H, vertical forces V, and moments M are balanced across the entire slice ($\sum H=\sum V=\sum M=0$). This condition ignores the reality, as shown in Figure 3.1.2, that forces acting on part of the moving layer (e.g., the end slice) are not transmitted 100% to the entire moving body and do not propagate to the fixed layer. In soil, due to soil plasticity and the conservation of energy, 100% of the load acting on the end slice is not transmitted to the upstream slice. The inter-slice forces calculated from the overall force equilibrium equation for the slice are fictitious forces that do not exist in actual ground.

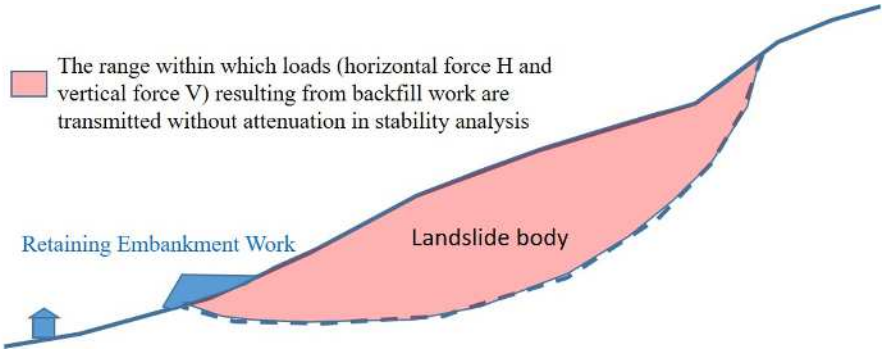


Figure 3.1.2 Range of increased in-situ soil stresses assumed by the stability analysis using the retaining embankment method in the limit equilibrium approach

The load applied to a single point in the ground by ground anchor works or similar device is transmitted radially outward from the loading point, as shown in Figure 3.1.3, without distinction between the mobile layer and the fixed layer. However, in the limit equilibrium method, the load is assumed to be transmitted without attenuation solely through the moving body, and analysis is performed under the assumption that the load is in equilibrium within the moving body. While the Fellenius method, which does not account for inter-slice forces, cannot evaluate this, analyzing the state shown in Figure 3.1.2 using methods that handle inter-slice forces, such as the Spencer method or the M&P method, shows that 100% of the anchor force is replaced by inter-slice forces. This supports the assumption that the load (anchor force) acting on a portion of the moving body is transferred to the entire mobile layer and is not transferred to the fixed layer.

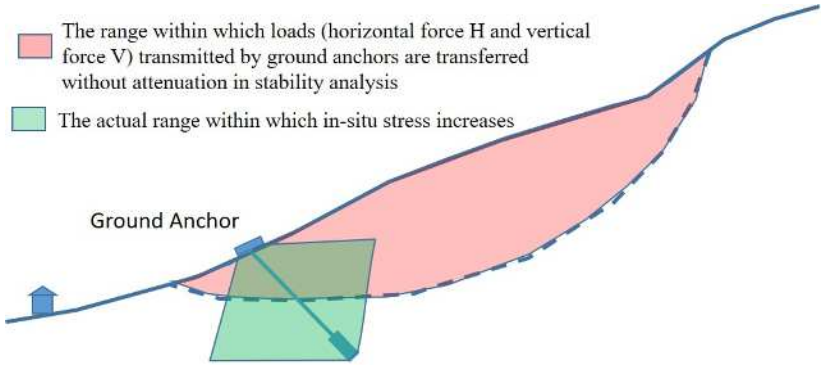


Figure 3.1.3 Transmission of concentrated forces at the ground surface and loads transmitted to the anchor body within the ground

2) Issues of the Strict Limit Equilibrium Method

The dashed and dotted lines in Figure 3.1.4(a) are examples of thrust lines required by the strict limit equilibrium method. This thrust line represents the point of action of the resultant force acting on the side of the fragment in Figure 2.2.3(b). The position of the thrust line only needs to be within the middle third of the soil mass height. Therefore, in methods like Spencer's method or M and P method, the safety factor varies depending on the position of the thrust line. For example, the safety factor F is calculated with a range, such as F = 1.24 to 1.36. It is impossible to determine which safety factor within this range is the true solution. This thrust line originates from the structural mechanics concept of force equilibrium ($\sum H = \sum V = \sum M = 0$) and does not exist in actual slopes. Because the safety factor varies with the thrust line position, different analysis software may calculate different safety factors. Furthermore, when using the Spencer method or M and P method for designing restraint measures like piles or ground anchor works, it becomes necessary to perform equilibrium calculations for the inter-slice forces that change due to the installation of these restraint measures, complicating the design calculations. This leads to unnecessary computational time spent calculating non-existent, hypothetical inter-slice forces and the unreasonable situation where stability calculation results differ between design software packages. Overseas, software utilizing advanced limit equilibrium methods like the Spencer method for 3D stability analysis is commercially available, but it faces similar issues.

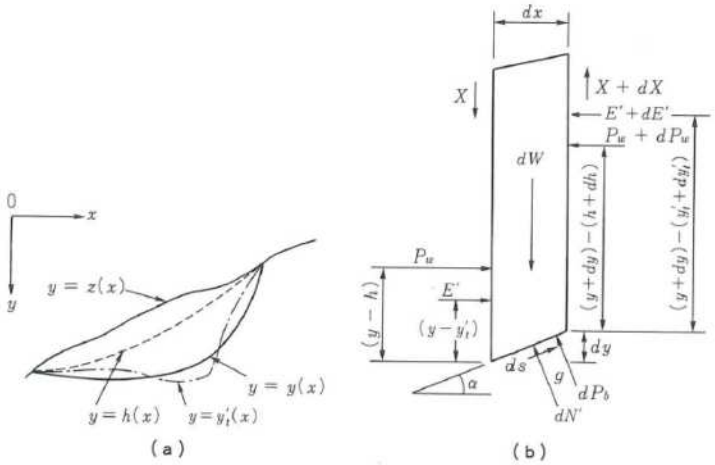


Figure 3.1.4 Internal forces considered in the Morgenstern-Price method (a): thrust line, (b): forces acting on the sliver (Shin, 1989)

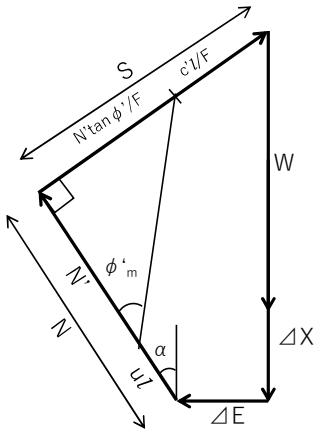
3) Issues with the Fellenius and Modified Fellenius Formulas

The Fellenius and Modified Fellenius formulas, widely used in Japan, are expressions that ignore the equilibrium of forces within a slice (force polygon, force diagram) as shown in Figure 3.1.5. Consequently, as the slip surface inclination angle α increases, the normal force on the slip surface ($W \cdot \cos\alpha$) is underestimated. For example, the normal force on a slip surface with an inclination angle α close to 90° is calculated as nearly zero. However, in actual ground conditions, even such steep slip surfaces experience sufficiently large static earth pressures. Consequently, applying the Fellenius or Modified Fellenius formula to steep slope failures underestimates the normal force on the slip surface, resulting in an overly conservative safety factor. Consequently, the cohesion c and internal friction angle φ calculated via inverse analysis are likely to be overestimated, potentially leading to an overestimation of the effectiveness of countermeasures. The simplified Bishop and simplified Janbu methods consider the equilibrium of forces within the slice (force polygon). Therefore, even for

steep slip surface inclinations, the normal force on the slip surface is not underestimated (Enokida, 2022).

Among internationally renowned software used overseas for slope stabilization design is SLOPE/W by Canada's Geo-SLOPE Ltd. (owned by Bentley Systems, Inc. as of 2022). Its technical manual concludes the explanation of the Fellenius formula with the statement: “The Ordinary method should not be used in practice, due to potential unrealistic factors of safety.” This indicates that the Fellenius formula, containing unrealistic factors, should not be used in practice. Consequently, the use of the Fellenius formula or modified Fellenius formula is not recommended for slope stabilization projects overseas. As simplified limit equilibrium methods, the simplified Bishop formula and simplified Janbu formula are more suitable for practical slope stabilization design.

However, given the extensive track record of the Fellenius and modified Fellenius methods in countermeasures for steep slopes in Japan, they can be utilized as an indicator to evaluate changes in the relative safety factor achieved by countermeasures. This is applicable specifically for circular slide, provided it is combined with appropriate shear strength evaluation (particularly φ value) based on soil testing and inverse analysis.



S: Shear force on the slip surface
N: Normal force on the slip surface
u: Pore water pressure
α: Angle of inclination of the slip surface
⊿*X*: Difference in vertical force between fragments
⊿*E*: Difference in horizontal force between fragments
Shear force on the slip surface

Figure 3.1.5 Force diagram acting on the slice (Enokida, 2022)

(3) Actions to Take

Rather than using methods like Spencer or M and P, which yield multiple safety factors depending on thrust line position even with identical analysis sections and soil strength parameters, it is preferable to design countermeasures such as restraint measures using simplified Bishop or simplified Janbu methods. These methods feature simpler slope geometry and analysis, clearer theory, and focus solely on the accumulation of relative safety factors. The simplified Bishop and simplified Janbu methods possess sufficient practicality for evaluating changes in relative safety factors due to the placement and quantity of countermeasures.

The simplified Bishop and simplified Janbu methods can also be used for forward analysis. Using the Fellenius method for forward analysis carries the risk of calculating an insufficient safety factor.

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3. 1. 3 Application of Inverse Methods in Setting Soil Strength Parameters

(1) Issue

In Japan, even for steep slope failure countermeasure projects, after determining the minimum safety factor for sliding, the soil strength parameters (c, φ) are adjusted so that the current safety factor F_s becomes $F_s = 1.0$. This is a type of inverse calculation method. For slow-moving landslide countermeasure projects, a method is adopted where the current safety factor is estimated based on the landslide's sliding conditions, and then c and φ are calculated inversely. Compared to Japan, Europe and the United States have simpler geological conditions and fewer triggers like torrential rains. Consequently, forward analysis is common even for slope failure countermeasures in these regions. Some commercially available English-language software sold in Europe and the US lacks backward analysis functionality. Therefore, many engineers in developing countries who routinely use Western design software question the necessity of backward analysis. Furthermore, in developing countries where natural slope disaster prevention measures are not implemented, slope stability analysis is frequently used to evaluate the stability of artificial ground such as embankments. For embankments, forward analysis using c and φ values from soil tests to calculate the safety factor is often employed, which is another factor contributing to skepticism about backward analysis.

(2) Explanation

1) Conditions for Applying Inverse Analysis

The prerequisite for applying inverse analysis in Japan's slow-moving landslide countermeasure projects and steep slope collapse countermeasure projects is that “the target slow-moving landslide area or slope has a risk of sliding, and the necessity for implementing some countermeasure has been clearly established.” Generally, Japan's geological distribution and structure are complex, resulting in heterogeneous ground strength within natural slopes. Slope failures originating from bedding planes or joint planes also occur frequently. Consequently, accurately predicting the risk of slope failure using forward analysis for stability assessment is difficult. Triggers such as torrential rain and earthquakes are also numerous, leading to frequent slope failures and reactivated slow-moving landslides nationwide. Therefore, engineers conduct on-site inspections to assess slope failure risk. If the risk is deemed high, they perform stability analysis using inverse analysis and consider countermeasures. Furthermore, applying forward analysis to reactivated slow-moving landslides often yields a calculated safety factor F that is inconsistent with the slope's actual movement observed through dynamic monitoring. Specifically, it is frequently observed that even when the calculated safety factor F is $F \gg 1.0$, landslide movement is confirmed, or conversely, when the calculated value is $F \ll 1.0$, no landslide movement is observed at all. This proves that the safety factor calculated by forward analysis is incorrect. Therefore, in slow-moving landslide countermeasure projects targeting reactivated slow-moving landslides, soil parameters are set using inverse analysis to ensure the calculated safety factor aligns with the slope's actual movement status.

In summary, since inverse analysis is a method premised on implementing countermeasures, it is suitable when geological distribution and structure are complex, and the frequency and magnitude of triggers like torrential rain or earthquakes are high.

2) Conditions for Applying Forward Analysis

Forward analysis is an analytical method that does not presuppose the implementation of countermeasures.

Applying backward analysis to slopes judged to be stable with no risk of failure, or to dormant landslide showing no signs of movement even over long periods, will inevitably yield results indicating that countermeasures are necessary due to the difference between the current safety factor and the design safety factor. Forward analysis is appropriate for such conditions where slope failure or slow-moving landslides are unlikely to occur.

For example, forward analysis is the mainstream method for stability analysis on cut slopes in low-rainfall regions of the United States, where slope failures or other deformations generally do not occur. Municipal standards typically require soil parameters to be determined through soil testing.

Similarly, forward analysis is sometimes used in Japan for preliminary stability assessments of road development cut slopes. However, the *Guidelines for Highway Earthwork - Cut Slope Protection and Slope Stabilization* (JRA, 2009) states in the section on the fundamental approach to cut work that conducting stability evaluations for individual slopes is the principle. Yet, in the section detailing stability calculations for cut work, it explicitly states that stability calculations are generally not performed, except for cuts in slow-moving landslide areas or on former collapse sites. The reason for this is that Japanese rock masses are extremely complex and heterogeneous, containing geological discontinuities and weathered or altered zones. Consequently, meaningful stability analysis cannot be performed accurately without precisely determining the distribution of ground parameters within the rock mass. However, stability analysis using forward analysis is possible when the rock mass consists of relatively homogeneous geology, as described below (JRA, 2009).

a) When surface collapse of topsoil is anticipated
b) When thick deposits of nearly homogeneous geological formations such as sand layers, talus deposits, colluvial deposits, gravel, or volcanic ash soil are present, and large-scale collapses or slow-moving landslide-type failures are anticipated within these deposits
c) When the natural ground at the analysis location is geotechnically nearly homogeneous, and it is determined that geological structures such as bedding planes or joint planes, or crack distributions, will not affect stability.

In summary, forward analysis is applicable when the geology is reasonably homogeneous, and it is particularly suitable when the frequency and magnitude of triggering events such as torrential rainfall or earthquakes are low. However, the trend in global design software in Europe and the United States is for forward analysis to be conducted concurrently with reliability design, including probabilistic evaluation of soil parameters and calculation of failure probabilities.

(3) Actions to Take

It is necessary to investigate the actual state of slope disasters in the target region of the country and to properly evaluate the slope disaster risk based on the survey results at the target site. When slopes and slow-moving landslide areas targeted for overseas landslide countermeasures or steep slope collapse countermeasures are assessed as having a high potential for slope movement, as explained in the above commentary, inverse analysis in the design of slope countermeasures is crucial. It is essential to ensure that engineers in the partner country, who are accustomed to forward analysis, understand this point.

For overseas road development projects, residential land development projects, or similar initiatives requiring prior stability assessment of cut slopes, forward analysis may be performed. When conducting forward analysis,

it is advisable to use Western design software compatible with reliability-based design.

3. 1. 4 Japanese Practices in Soil Parameter Setting

(1) Issue

In Japan's slow-moving landslide countermeasure projects, it is common practice to set the cohesion c based on the maximum depth h_{max} or average depth h of the slip surface, and then calculate the internal friction angle φ. Table 3.1.1 shows the relationship between maximum vertical layer thickness and cohesion. Tables with identical figures but using average vertical layer thickness instead of maximum vertical layer thickness also exist.

Table 3.1.1 Relationship between Maximum Vertical Layer Thickness and Cohesion c of Slip Surface Clay Used in Japan

Maximum vertical layer thickness of landslide mass (m)	Cohesion c (kN/m²)
5	5
10	10
15	15
20	20
25	25

In recent years, this method has also been used in stability analysis for slope failure countermeasures in Japan. Few engineers can explain the basis for this approach. When using this method overseas, it becomes necessary to explain its theoretical foundation.

Furthermore, in Japan, technical manuals such as successive editions of the *Guidelines for Highway Earthwork* (e.g., JRA, 2009) and *Disaster Recovery Handbook* (e.g., JCEA, 2025) specify a default value of 18 kN/m³ for the unit weight of the moving layer (γ_t) in slow-moving landslides. However, few engineers understand the basis for this value. This value is significantly lower than the actual unit weight of soils found in Japan's mountainous regions (weathered rock and gravelly soil). Yet, because it is listed in Japanese technical standards, it has become the standard value in Japan. Underestimating the unit weight leads to underestimating the sliding force in landslides and the required stabilizing force for restraint measures, increasing the risk of under-designed these structures. Furthermore, directly applying Japan's standard unit weight value overseas, where geological conditions differ, is problematic.

(2) Explanation

a) Basis and Issues of the Method Setting Cohesion Based on Slip Surface Depth

It was known even before 1975 that the slip surface strength of reactivated slow-moving landslides, which are the target of slow-moving landslide countermeasure projects, reaches residual strength, and that the cohesion $\boldsymbol{c}$ as residual strength is nearly zero. However, setting cohesion $\boldsymbol{c} = 0$ in slow-moving landslide countermeasure projects caused problems in flood-prone slope countermeasures and earthwork design, necessitating a method setting $\boldsymbol{c} > 0$. This led to attention being focused on the research results of Yoshioka and Ito (1971).

Yoshioka and Ito (1971) presented analysis results regarding the setting values for soil strength parameters on slow-moving landslide slopes, based on questionnaire survey findings concerning slow-moving landslide countermeasure projects at that time. Figure 3.1.6 shows the relationship diagram between maximum landslide depth h_{max} and cohesion $\boldsymbol{c}$ (Figure 5(a) in the original source) published by Yoshioka and Ito (1971). Although

Yoshioka and Ito noted in their discussion of this diagram that no significant trend was apparent, the diagram later drew attention at the Public Works Research Institute (PWRI). Consequently, a table for setting cohesion based on slip surface depth was included in "Chapter 13: Slow-moving Landslide Prevention Facility Planning" of the *Ministry of Construction (MOC), Technical Criteria for River and Erosion Control Works (Draft) - Planning Volume* (MOC, 1976), published in 1976. The regression equation for this origin-return calculation is $Y = 0.102 \cdot X$, relating $\boldsymbol{c}' \approx 0.1 h_{max}$. This became the basis for estimating cohesion $\boldsymbol{c}'$ from slip surface depth h_{max}. However, the coefficient of determination R^2 for this regression calculation is approximately -0.17, indicating the low reliability of the method based on slip surface depth. Thus, the method of setting cohesion $\boldsymbol{c}$ based on slip surface depth originated from the administrative circumstances of Japan's slow-moving landslide countermeasure projects, where $\boldsymbol{c}$ was set to be greater than zero ($\boldsymbol{c} > 0$) for reactivated slow-moving landslides, which inherently have $\boldsymbol{c} \approx 0$.

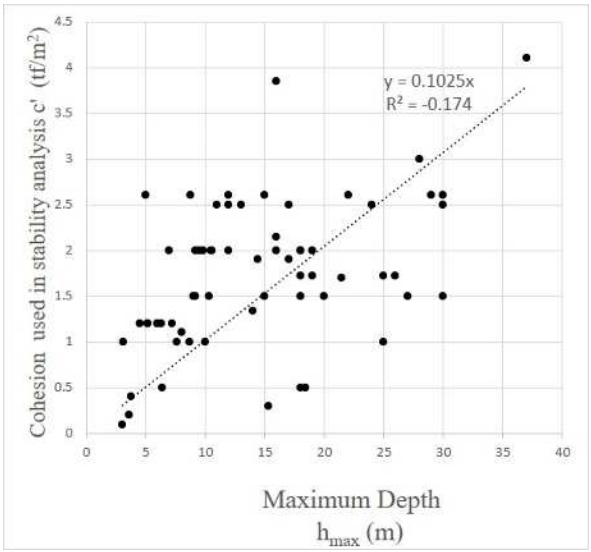


Figure 3.1.6 Relationship between maximum slip depth h_{max} and cohesion c' (Yoshioka and Ito, 1971, reproduced including plotting errors)

Furthermore, the original data from Yoshioka and Ito (1971) represent soil strength parameters adopted in slow-moving landslide countermeasure projects at the time of 1971, and naturally include results from inverse analysis. There is no confirmation that these represent the true soil strength parameters of the slow-moving landslide sites in question. Conversely, the large variation in Yoshioka and Ito's graph indicates that many slow-moving landslide countermeasure projects in 1971 did not adopt cohesion proportional to slip surface depth.

Some technical standards adopt a method of setting cohesion using average depth instead of maximum depth of the slip surface. However, this method is derived from the method using maximum depth and lacks geotechnical justification.

This method originated in an era when techniques for sampling slope-face clay were limited, and tests like the ring shear test to determine residual strength of slope-face clay were not widely adopted. Ideally, the internal friction angle φ for residual strength should be determined via ring shear tests or similar methods, and cohesion c should then be calculated inversely.

When setting the cohesion c first and then back-calculating the internal friction angle φ, the resulting φ incorporates various errors inherent in the stability analysis formula (such as errors from analyzing 3D phenomena

in 2D or computational errors inherent in the stability analysis method itself). This error is then carried into the evaluation of the effectiveness of ground anchor shear-resistance increasing effect and groundwater drainage works, where the magnitude of φ directly affects the assessment results.

b) Issues in Slope Failure Countermeasures for Steep Slopes and Road cut slopes

In slope failure mitigation for steep slopes, it is common practice to adjust c and φ so that the safety factor for the most critical slip plane matches the current safety factor F (typically $F = 1.0$). This constitutes inverse analysis. Recently, engineers accustomed to methods for setting cohesion based on slip plane depth in Japanese slow-moving landslide countermeasure projects have begun applying this approach to inverse analysis for slope failure countermeasure as well.

For preemptive slope failure countermeasures, stability calculations typically target bare rock where no existing slip surface is present. Since the soil strength parameter required for stability analysis is the peak strength of the bare rock, and the assumed slip surface is shallow, field tests like the simplified penetration test can be used. Ideally, inverse analysis should be performed using soil strength parameters actually confirmed by such methods. Equipment for simplified penetration tests, for example, can be easily transported overseas.

c) Background and Issues in Setting Soil Unit Weight

The *Guidelines for Highway Earthwork* (JRA,1956) contain no description of soil unit weight. The *Design Guidelines for Expressways* by JH (1964) included the unit weight values shown in Table 3.1.2. For natural ground within this table, a value of 1.8 tf/m³ (=1800 kgf/m³) was adopted for hard cohesive soils and unconsolidated gravel or sand.

Table 3.1.2 Unit Weight of Soils Listed in Table 4-1 of the *Design Guidelines for Expressways* by JH (1964)

Type		Condition		Moist Unit Weight (tf/m³)*
Embankment	Gravelly sand	Compacted		2.0
	Sand	Compacted	Well-graded	2.0
			Poorly graded	1.9
	Sandy soil	Compacted		1.8
	Cohesive soil	Compacted		1.7
	Kanto loam	Compacted		1.4
Natural ground	Gravel	Dense or well-graded		2.0
		Non-dense or poorly graded material		1.8
	Gravelly sand	Dense material		2.1
		Non-dense material		1.9
	Sand	Dense material or well-graded material		2.0
		Non-dense or poorly graded material		1.8
	Sandy soil	Dense material		1.9
		Loose material		1.7
	Cohesive soil	Hard material		1.8
		Soft material		1.6
	Silt	Hard material		1.6
		Soft material		1.4
	Clay	Hard material		1.7
		Soft material		1.5
	Kanto loam			1.4

(Note: 1.0 tf=1000 kgf)

Guidelines for Highway Earthwork: Temporary Structures (JRA, 1999) adopts the values in Table 3.1.3 as the unit weight of soil. While Table 3.1.3 assumes natural ground, backfill soil varies depending on compaction methods, etc., so the actual weight is used as a principle. For reference in calculating earth pressure, 18 kN/m³ is explicitly stated.

However, in slow-moving landslide countermeasure projects and steep slope failure countermeasure projects, the ground often consists of weathered rock or gravelly soil. The actual weight is frequently equivalent to or greater than the dense gravelly soil value in this table.

Table 3.1.3 Moist Unit Weight of Soil (JRA, 1999)

Soil Type	Dense (kN/m³ (tf/m³))	Loose (kN/m³ (tf/m³))
Gravelly Soil	20 (2.0)	18 (1.8)
Sandy Soil	19 (1.9)	17 (1.7)
Cohesive Soil	18 (1.8)	14 (1.4)

(Note: 1.0 tf=1000 kgf)

According to the results of interviews regarding the basis for setting the old standard values: "In the past, we did not use core boxes but instead took samples and conducted various soil tests. The data from those tests yielded a value for the moving mass of landslide in colluvial deposits of roughly this amount (1.8tf/m³ = 1800 kgf/m³). Thus, in the past, we proceeded using soil tests and a straightforward approach (forward analysis), but (regarding stability analysis) the results were not favorable. Later, soil tests were discontinued because the test values were deemed unrealistic. However, since the unit weight of 1.8 tf/m³ caused no issues, it continued to be used." Additionally, there is testimony stating, “Although it was known that some slow-moving landslide countermeasure projects had unit weights greater than 1.8 tf/m³, the standard of 1.8 tf/m³ was maintained because increasing the unit weight would require more restraint measures and other works, thereby increasing the project volume.” Recently, with improved core sampling rates, it has become common to measure the weight of the core box to determine the actual unit weight.

(3) Actions to Take

When calculating the adhesion coefficient c and the angle of internal friction φ, it is important to set the angle of internal friction φ appropriately, as it significantly affects the effectiveness of groundwater dewatering measures and the shear-resistance increasing effect of ground anchor works.

The method of setting cohesion c based on slip surface depth is unique to Japan and is not used overseas due to the lack of logical justification, such as in soil mechanics. The standard method involves estimating the internal friction angle φ from sounding tests or soil tests and then back-calculating the cohesion c. However, overestimating cohesion c risks overestimating the effectiveness of soil removal works. Therefore, c and φ must be set to appropriate values, considering their impact on the evaluation of other countermeasure effectiveness.

Regarding unit weight, if the partner country has standard values for each soil type, those should be adopted. If the partner country lacks standard values, Japanese standard values based on soil type, as shown in Table 3.1.1, should be referenced for setting. For rock slopes containing weathered rock, separate consideration based on the specific geology is required.

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3. 1. 5 Japanese Practices in Setting Initial Safety Factors and Design Safety Factors

(1) Issues

In Japan's slow-moving landslide countermeasure projects, the initial safety factor F_0 = 0.95 to 1.00 and the design safety factor F_p = 1.05 to 1.20 are standard. Even for collapse countermeasures on steep slopes, a design safety factor F_p = 1.20 is generally applied. Some countries overseas set the slope design safety factor at F_p = 1.5. When applying Japan's standard values overseas, the basis for setting them must be demonstrated.

(2) Explanation

Japan's slow-moving landslide countermeasure projects and steep slope failure countermeasure projects are fundamentally based on inverse analysis, emphasizing the relative difference between the initial safety factor and the design safety factor.

Regarding the basis for setting the reference value, the interviews revealed that the practice of setting the initial safety factor F_0 for continuously sliding landslides at F_0 = 0.95 became established. This stems from several experiences in Japan's slow-moving landslide countermeasure projects. Specifically, when the safety factor F was set to F = 1.0 at the groundwater level when the landslide began moving, subsequent increases in groundwater level resulted in a reduction in the safety factor to approximately F = 0.95.

At the start of a project, since data such as dynamic monitoring and groundwater level observations are unavailable, an initial safety factor (0.95 to 1.0) considering the slow-moving landslide's activity status should be adopted. The most reasonable approach is to review both the initial safety factor and the slip surface strength parameters (c, φ) when the critical water level is confirmed during the project's intermediate years.

The planned safety factor F_p = 1.05–1.2 used in Japanese slow-moving landslide countermeasure projects is an empirical value considering factors like countermeasure costs. The interviews revealed responses such as, “For slow-moving landslide countermeasure projects targeting reactivated slow-moving landslides, the project cost ceiling is around a planned safety factor F_p = 1.2. Setting a higher planned safety factor would result in prohibitively high project costs, making implementation unfeasible.” For steep slope failure countermeasure projects, F_p=1.5 may be used when the protected area is particularly critical, but in such cases, feasibility in terms of cost must be evaluated. In any case, the design safety factor Fp adopted in Japan is a relative value derived from the initial safety factor F_0. It is adopted based on the proven track record that achieving this design safety factor through a combination of countermeasures stabilizes reactivated slow-moving landslides.

In countries overseas that primarily use forward analysis, larger design safety factors than Japan's are often adopted, such as F_p=1.5. In Europe and America, comprehensive countermeasure projects are often not implemented for large-scale reactivated slow-moving landslides. Therefore, setting the design safety factor to F_p=1.5 applies to collapses and relatively small slow-moving landslides where realistic countermeasures are feasible. Furthermore, this reflects the consideration that soil samples tested often represent the most stable geology available for sampling, not necessarily the most vulnerable within the site. Forward analysis thus carries the risk of calculating a safety factor larger than the actual value.

Because forward analysis is influenced by sampling in this way, many global design software programs in Europe and the United States that support forward analysis evaluate the effectiveness of countermeasures by considering both the change in failure probability due to the countermeasures and the increase in the safety factor. They do this by using a slope failure probability based on a probabilistic evaluation that accounts for the variability

of soil strength parameters.

(3) Actions to Take

The initial safety factor for projects adopting inverse analysis is crucial as an indicator at the project's commencement. Similar to Japan, it is advisable to determine and set the initial safety factor based on the activity status of slow-moving landslides and the target slope. When dynamic monitoring or groundwater level observation is conducted, the current safety factor must be reassessed based on slow-moving landslide activity during the period of highest water levels. In countries like Japan where slope failures are frequent, inverse analysis is also suitable for slope failure countermeasures. In such cases, the initial safety factor should be set using the Japanese methodology. In countries with fewer slope failures where forward analysis is appropriate, setting an initial safety factor is unnecessary.

Regarding the design safety factor, Japan's practical experience should be explained, and it is desirable to set it according to the importance of the protected structure, similar to Japan. Even when slope failures are the target, and the country's practices or technical standards are based on forward analysis and a design safety factor F_p=1.5, in countries where slope failures frequently occur, backward analysis, which is more suitable for evaluating countermeasures, should be recommended. In such cases, Japan's standard value (F_p=1.2) is appropriate as the design safety factor. Adopting a design safety factor of F_p=1.5 is also possible when particularly important structures exist. Forward analysis may be adopted when conditions suitable for its application are met, such as low rainfall and homogeneous geology. In such cases, it is desirable to use global design software from Europe and America that incorporates probabilistic evaluation of soil strength parameters.

3. 2 Design Considerations

3. 2. 1 Stability Analysis Methods According to Landslide Classification

Table 3.2.1 Classification of Slides, Approach to Countermeasure Projects, and Slope Stability Analysis Methods

Detailed Classification	Approach to Countermeasure Projects	Slope Stability Analysis Method
Reactivated slow-moving landslide (slow-moving slide) ※1) A natural phenomenon where part of the land slides due to groundwater, etc., or a natural phenomenon involving movement associated with this ※2) Slow-moving Landslide Countermeasure Projects	Post-event countermeasures are fundamental. Pre-emptive countermeasures are implemented only when particularly important conservation targets exist. Since reactivated slow-moving landslides tend to be relatively large-scale, the project cost per site also becomes significant. Therefore, countermeasures are typically not implemented for slow-moving landslides that have not slid in recent years (e.g., old dormant landslide). Since the movement speed is slower than that of collapse and involves repeated sliding and pausing, it is desirable to conduct various surveys to understand the distribution of slip surfaces, as well as long-term observations of groundwater levels and displacement amounts. For slow-moving landslides where sliding is confirmed through dynamic monitoring, the necessity of countermeasures is examined. For primary landslides such as triggered by earthquakes, countermeasures are considered if there is a risk of re-sliding or after re-sliding has been confirmed. However, preventive measures may be implemented in advance when road development involves excavating the terminus of a dormant landslide or when dam impoundment is predicted to destabilize the landslide terrain.	Survey the existing slip surface location and perform stability analysis on the existing slip surface geometry at the analysis section.
Slope failures (collapses) (rapid slide) ※1) A natural phenomenon where land with a slope gradient of thirty degrees or more collapses. ※2) Steep Slope failure Countermeasure Project	The basic approach involves identifying hazardous areas and implementing preventive measures. Post-collapse countermeasures are also implemented. Since high movement velocity often becomes high, protectible structures are frequently damaged immediately after a collapse occurs. Post-collapse countermeasures cannot function to prevent damage. Since existing slip surfaces are often not yet formed, dynamic monitoring and similar surveys are not conducted. Instead, the slope gradient, height, and surface conditions are used to assess the hazard and determine the necessity of countermeasures.	For preventive measures, since there is no defined slip surface, one must either infer the depth at which a slip surface could form based on nearby collapse cases or detect the arc of minimum safety factor and the arc of maximum stabilizing force to perform a stability analysis. For post-collapse measures, stability analysis is performed using the same method as for reactivated slow-moving landslides. Design countermeasures to exceed the planned safety factor for all possible slides that could occur on the target slope.

※1) Definitions are based on Japan's "Slow-moving Landslide Prevention Act" and Article 2 of the "Act on Promotion of Measures to Prevent Sediment Disasters in Sediment Disaster Warning Areas, etc." However, the Japanese term "jisuberi" follows the former.
※2) Corresponding main public works project names in Japan.

Academically, landslides are classified by movement patterns such as "fall," "topple," "slide," "spread," "flow,"

and “creep.” In the actual design and construction of slope protection works, the ‘slide’ movement pattern is further subdivided based on phenomenon characteristics—such as collapse, reactivated slow-moving landslides, and deep-seated slope failures—while “fall” is categorized as rockfall or rock mass collapse. This classification considers the need for appropriate planning and design of countermeasures. The approach to slope disaster prevention projects in Japan differs depending on the phenomenon. For example, the project implementation methods and design concepts for countermeasures differ significantly between collapses (steep slope failure countermeasures) and reactivated slow-moving landslides. Table 3.2.1 shows these differences.

3. 2. 2 Differences in Stability Analysis for Steep Slope Collapse Countermeasures and Reactivated Slow-moving Landslide Countermeasures

As explained in Section 3.2.1, even for landslides classified under the same movement pattern in Japan, countermeasures differ between reactivated slow-moving landslides and steep slope failures.

Reactivated slow-moving landslides, which alternate between sliding and resting, present distinct issues: a) 1. The scale of the moving mass is relatively large, making prevention measures costly. 2. More accurate understanding of the moving mass's shape allows for effective calculation of the placement and quantity of countermeasures. 3. The movement speed is relatively slow, allowing sufficient effectiveness even if countermeasures are implemented after understanding the landslide's activity characteristics. Therefore, countermeasures are considered after investigating the movement range and slip surface depth.

Figure 3.2.1 illustrates the role of slope stability analysis in the design of countermeasures for reactivated slow-moving landslides.

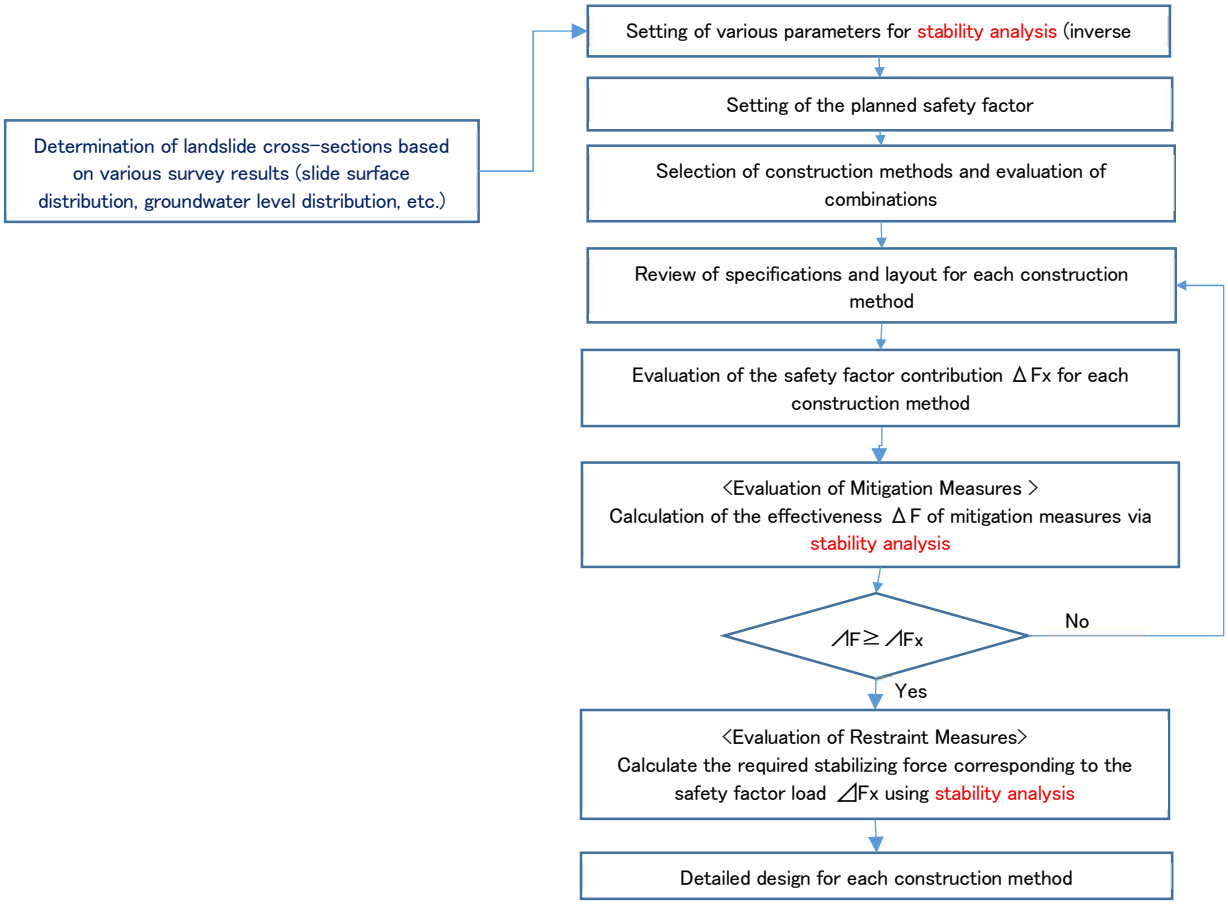


Figure 3.2.1: Positioning of Slope Stability Analysis in Countermeasure Design for Reactivated Slow-moving Landslides

On the other hand, in steep slope countermeasure projects, slope movement occurs rapidly, and once movement is detected, a disaster is highly likely to occur immediately. Therefore, preemptive measures before slope movement occurs are crucial. The slope movement phenomena addressed by steep slope countermeasures include rockfall, rock mass failure, and collapses, each requiring different stability analysis methods. Among these, collapses use the same stability analysis equations as slow-moving landslides, but the analysis process differs because no existing slip surface exists at the design stage. Figure 3.2.2 shows the positioning of slope stability analysis in steep slope failure countermeasures targeting collapses .

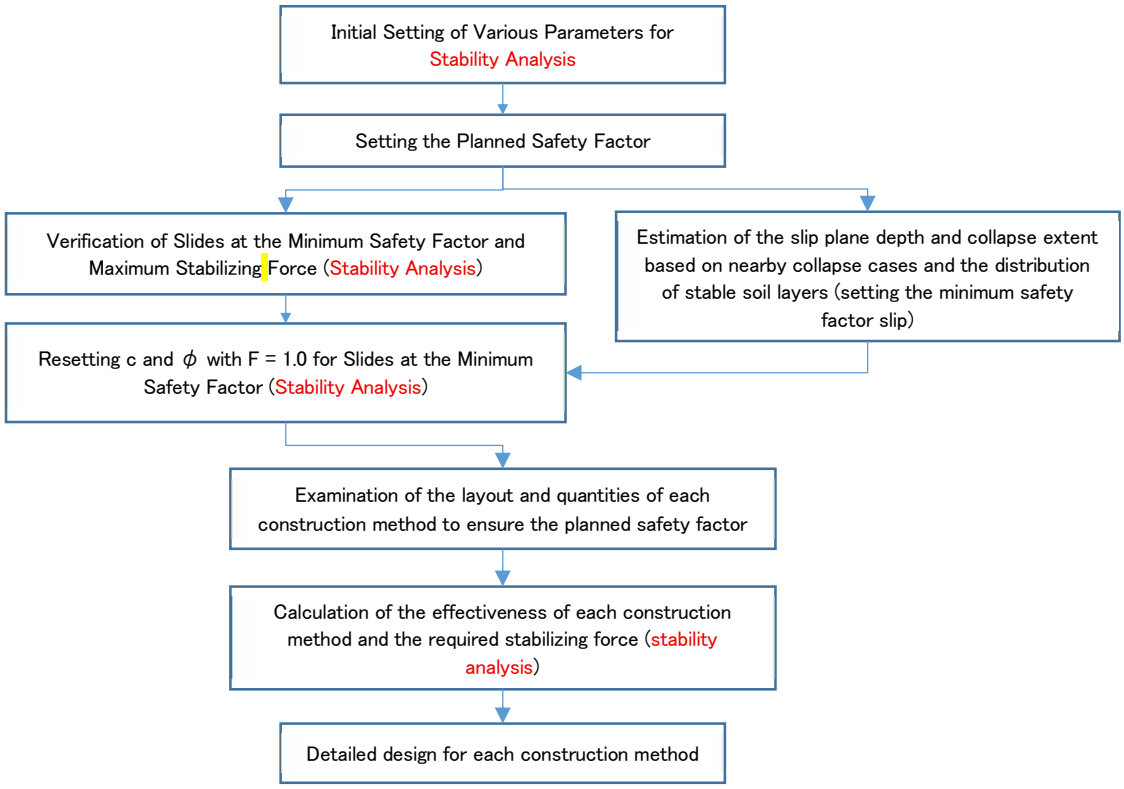
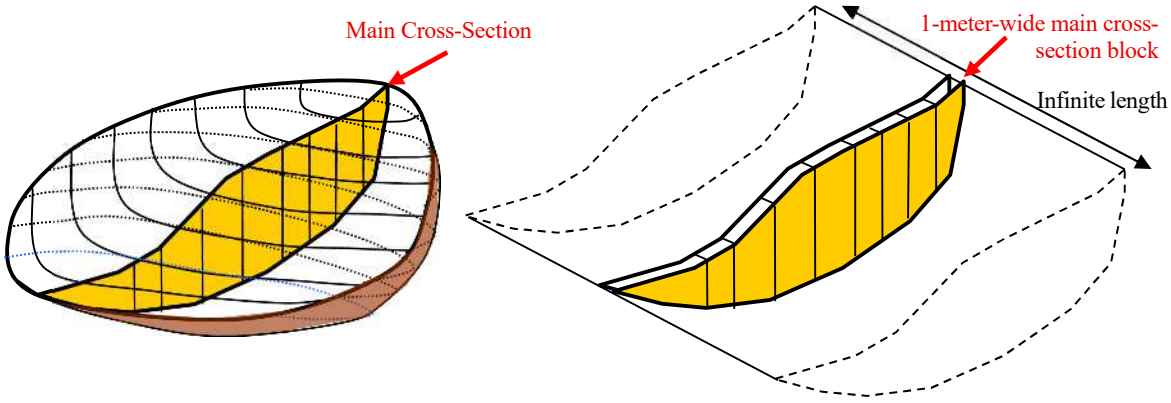


Figure 3.2.2: Role of Slope Stability Analysis in Steep Slope Failure Countermeasures

3. 2. 3 2D Stability Analysis and 3D Stability Analysis

As shown in Figure 3.2.3(a), the moving mass in reactivated slow-moving landslides and slope failures consists of three-dimensional blocks of soil or rock. However, two-dimensional stability analysis is primarily employed, focusing solely on the shape of the main cross-section and dividing the main cross-section block into small fragments per unit width (typically 1 m) for analysis. This method calculates the safety factor based on the assumption that the moving mass of a reactivated slow-moving landslide or slope failure consists of an infinite number of blocks on either side, each having the same shape as the main cross-section, as shown in Figure 3.2.3(b). Consequently, the three-dimensional shape of the moving mass being analyzed differs from the actual one.



(a) Example of the 3D shape of a landslide

(b) Shape of the landslide mass assumed in 2D stability analysis

Figure 3.2.3: Relationship between the shape of a landslide mass (e.g., reactivated slow-moving landslide) and 2D stability analysis

Overseas, commercial software for 3D stability analysis using advanced limit equilibrium methods such as the Spencer method is available. As mentioned earlier, since the limit equilibrium method itself is a theory that ignores the actual properties of the ground in the natural terrain, even adopting 3D stability analysis cannot yield a true safety factor. The issues with advanced limit equilibrium methods mentioned in Section 3.1.2 apply equally to 3D stability analysis. Some viewpoints suggest that various computational errors increase compared to 2D stability analysis. Nevertheless, the primary benefits of adopting 3D stability analysis are as follows:

＜Key Benefits of Using 3D Stability Analysis＞

1) It allows for a more appropriate evaluation of the effectiveness of countermeasures, particularly the effectiveness of mitigation measures, compared to 2D stability analysis.

While 2D stability analysis cannot evaluate the effectiveness of restraining works (such as cut-and-fill works, embankment works, and groundwater drainage works) located far from the analysis section, 3D stability analysis makes this possible.

2) It can evaluate the three-dimensional effects of the moving mass to a certain extent.

While significant errors in capturing the three-dimensional shape of the moving mass can lead to increased calculation errors, if the three-dimensional shape of the moving mass can be represented with a certain degree of accuracy, it is possible to evaluate three-dimensional effects such as the edge effect at the sidewall of the moving soil mass to a certain extent. Since two-dimensional stability analysis cannot evaluate such three-dimensional effects at all, this is an advantage of three-dimensional stability analysis.

3) Introducing 3D stability analysis may reduce the amount of required countermeasures compared to 2D stability analysis.

3. 2. 4 Stability Analysis Considering Seismic Motion

One approach involves incorporating horizontal seismic coefficient k_h and vertical seismic coefficient k_v from the seismic coefficient method into the stability analysis formula based on the limit equilibrium method. Here, the seismic coefficients (k_h, k_v) can be calculated by dividing the maximum acceleration value corresponding to the seismic level classification (Level 1 to 3) by the gravitational acceleration (980 gal), or by using empirical formulas (such as Noda's formula) derived from structural damage conditions. The following is an empirical formula developed by Noda et al. (1975). He collected numerous examples of damaged and undamaged port structures and derived the relationship between the maximum acceleration of the seismic motion input to the structure and the design seismic coefficient.

$$k_H = \frac{1}{3}\left(\frac{A_{\max}}{g}\right)^{1/3}$$

Here, $A_{\max}$: Observed maximum acceleration, g : Gravitational acceleration

However, since seismic motion is a wave, acceleration in a specific direction does not act uniformly across the entire moving layer. For instance, while downstream acceleration may occur in part of the slow-moving landslide mass, upstream acceleration acts in another part on a slope. Therefore, even if the safety factor F_s in stability analysis considering horizontal seismic coefficient falls below 1.0, it is considered not to reflect the actual safety factor. It is not desirable to evaluate countermeasures solely based on the magnitude of the safety factor when using the seismic coefficient method, and this approach is generally not adopted in Japan either.

In Japan, the horizontal seismic coefficient used in the design of retaining walls and similar structures is specified in various technical standards. However, this represents the horizontal seismic coefficient acting on relatively small structures like retaining walls. The average horizontal seismic coefficient acting on an entire moving mass, which can extend tens to hundreds of meters in length, differs from the horizontal seismic coefficient acting on retaining walls. Therefore, applying the horizontal seismic coefficient for retaining walls to stability analyses for slope failures or reactivated slow-moving landslides results in an underestimated safety factor. In some target countries, a standard value for horizontal seismic coefficient used in the design of retaining walls may not be available.

References

Noda, S., Uwabe, T., and Chiba, T. (1975): Seismic Coefficient and Ground Acceleration of Gravity Quaywall, Report of the Port and Harbour Research Institute, Vol. 14, No. 4, pp. 67-111.

3. 2. 5 Advantages of the Limit Equilibrium Method and Issues in Utilizing Advanced Analysis Methods Such as the Finite Element Method

As previously mentioned, the limit equilibrium method has many inherent limitations. However, it remains the best available method for slope stability analysis in slow-moving landslide countermeasure projects at this time. The reasons are as follows:

1) It has over 60 years of proven application in Japan's slow-moving landslide countermeasure projects and has been used in the design of countermeasure works of various slow-moving landslide.
2) Its theory is relatively simple, making it easy for third parties to verify analysis results, including the parameters used.
3) While it cannot calculate the true slope safety factor, its effectiveness as a method for evaluating the relative change in safety factor due to countermeasures has been confirmed by its track record in Japanese landslide countermeasure projects.

On the other hand, several studies exist that perform slope stability analysis using advanced analytical methods such as the finite element method. However, employing such advanced methods present greater issues than using the limit equilibrium method.

1) The analytical results vary significantly depending on how the analytical model is constructed.
2) When the analytical model is made complex to match the actual soil conditions, such as using a nonlinear elastoplastic model, the analytical results differ depending on the specific analysis code used.
3) While inverse analysis excels at adjusting parameters to fit observed phenomena, forward analysis—where parameters are determined first—can yield results hundreds of times different from reality.
4) The complexity of setting analysis parameters and models makes it difficult for non-specialists to verify results.
5) There is no established method for determining analysis parameters from soil test values and obtaining favorable results through forward analysis.
6) When using the finite element method for public works projects, it is necessary to explicitly define the methodology: (a) fix the analysis code and model, and (b) forward-analyze the site's parameters from soil test values. However, the latter method (b) has not been finalized.

The finite element method is not typically used for slope stability analysis, but rather for analyzing slope deformation when deformation becomes a concern. However, a issue exists in pre-analysis for natural slopes composed of heterogeneous ground, where the reliability of the analysis results cannot be guaranteed.

However, in some countries, such as Indonesia and Malaysia, design software like Plaxis 2D/3D, which supports the finite element method, is widely used for slope protection. In this case, since the analysis code and model are fixed, some of the aforementioned issues are mitigated. For overseas projects in countries utilizing this software, there is a possibility that this design software will be employed.

3. 3 Example Descriptions of Items Requiring Attention When Developing Technical Standards

3. 3. 1 Selection of Stability Analysis Formulas (Countermeasures for Reactivated Slow-moving Landslides)

Select the stability analysis formula by considering factors such as the shape of the slip surface and the groundwater conditions on the target slope.

【Explanation】

1) Use of the Limit Equilibrium Method

The principle for stability analysis methods is to use the limit equilibrium method, which has fewer parameters and allows for easy verification of analysis results. The main parameters used in the limit equilibrium method include the unit weight of the moving body and the shear strength parameter of the slip surface, and the approximate validity of their values can be easily evaluated. The method of use during the design of countermeasures is also largely established.

The safety factor calculated by limit equilibrium slope stability analysis provides a rough indication of slope stability and is not the true safety factor of the slope. Therefore, in slope stabilization projects, emphasis is placed on the relative change in the safety factor resulting from the construction of the countermeasure works.

2) Selection of Slope Stability Analysis Formulas

Reactivated slow-moving landslides and collapses are three-dimensional mass movement phenomena. When the three-dimensional shape of the moving mass can be determined with relatively high accuracy, three-dimensional stability analysis can be used. Generally, two-dimensional stability analysis, which has a greater track record of use, is employed.

Two-dimensional stability analysis equations can be categorized into those for non-circular slides and those for circular one. When performing analysis, select the stability analysis equation that matches the longitudinal profile shape of the slip surface.

a) Non-circular slide case: Simplified Janbu method

b) Circular slide case: Simplified Bishop method, Fellenius method, Modified Fellenius method

However, when adopting the Fellenius method or Modified Fellenius method, attention must be paid to the contents described in the [Reference] section below.

These stability analysis formulas are generally designed for confined groundwater conditions. If the groundwater at the site is unconfined, hydrostatic pressure also acts on the slice faces, causing the form of the formula for pore water pressure within the stability analysis formula to differ slightly from that for confined groundwater.

【Reference Notes】

1) Why a Strict Solution Using the Limit Equilibrium Method is Unnecessary ?

The Morgenstern and Price formula and Spencer formula are stability analysis formulas that calculate more precise solutions as limit equilibrium methods. However, these are precise solutions within the assumptions of the limit equilibrium method, not precise solutions in the sense of calculating values close to the true safety factor of

the slope in question. The limit equilibrium method assumes unrealistic conditions, such as treating the moving body as a rigid body and ignoring load dispersion phenomena in the ground and the resulting stress attenuation. Consequently, it cannot calculate the true safety factor of the slope.

Furthermore, using a strictly accurate stability analysis formula can yield multiple solutions (safety factors) due to differences in the estimated thrust line position, making it impossible to limit the solution (safety factor) to a single value. Therefore, even with identical analysis parameters for the same analysis section, stability analysis results may differ depending on the design software used. Furthermore, there are disadvantages such as the complexity of precisely solving the equilibrium between the restraint forces of measures like pile or ground anchor works and the inter-slice forces when employing such restraint measures.

Since even the exact Janbu formula yields solutions that vary with thrust line position, a method fixing the thrust line at one-third of the slice height is often adopted. However, this approach becomes unstable for convergence calculations when adjacent slices exhibit large variations in slip surface inclination angle, such as when testing chair-shaped slip surfaces common in rock slides or overriding slip of ground anchor works. In such cases, calculation errors become significant. Therefore, for non-circular slides, the simplified Janbu formula is the optimal stability analysis equation.

2) Differences in Stability Analysis Formulas Due to Groundwater Pressure

Generally, groundwater acting on reactivated slow-moving landslides is considered to flow through fracture zones near the slip surface. Therefore, groundwater is judged to be either confined groundwater or unconfined groundwater, and only the water pressure acting on the slip surface is considered. Consequently, stability analysis formulas corresponding to confined groundwater are typically adopted. On the other hand, for stability analysis of collapses, the appropriate stability analysis formula is selected based on the groundwater conditions of the site under analysis: either the confined groundwater formula (considering only the hydraulic pressure on the slip surface) or the unconfined groundwater formula (considering the hydraulic pressure on both the slip surface and the flank).

3) Application of the Fellenius Formula and Modified Fellenius Formula

In Japan, the Fellenius formula or modified Fellenius formula is used as the stability analysis formula for circular slides in steep slope countermeasures. These stability analysis formulas are theories that ignore the equilibrium of forces within a slice (force polygon). Consequently, when the slip surface inclination angle becomes steep, they underestimate the normal force due to soil mass weight. Furthermore, when the slip surface inclination angle α approaches 90°, they evaluate the normal force due to soil mass weight as nearly zero. In reality, earth pressure still acts on such steep slip surfaces. Consequently, applying the Fellenius or modified Fellenius formula to steep slope failures leads to an underestimation of the normal force. This, in turn, results in an underestimation of the safety factor, causing the cohesion c or internal friction angle φ calculated by the inverse method to be overestimated. This overestimation may lead to an overestimation of the effectiveness of restraint measures. Furthermore, in Japan, the Fellenius formula and modified Fellenius formula have been extensively used since the 1980s for stability analysis of non-circular, reactivated slow-moving landslides. This is a mechanically incorrect convention. However, it is primarily because the stability analysis aims not to evaluate the true stability of the slow-moving landslide, but rather to assess the change in the safety factor due to countermeasures as a relative

change from the current state. The formula's simplicity and ease of understanding, along with the straightforward method for calculating the required stabilizing force for restraint measures, are also contributing factors to its frequent use.

The technical manual for SLOPE/W, a globally renowned design software for slope stabilization used overseas (developed by Canada's Geo-SLOPE Ltd., transferred to a U.S. company as of 2022), explains that the Fellenius formula should not be used in practice because it includes unrealistic factors.

Overseas, the simplified Bishop formula or simplified Janbu formula is preferred. If the Fellenius formula or modified Fellenius formula must be used, it should be limited to evaluating changes in the relative safety factor due to the slope protection structure using inverse analysis. It should not be used for slope stability assessment via forward analysis. Furthermore, applying the Fellenius formula or the similar formula to non-circular slides should absolutely be avoided.

3. 3. 2 Setting Current Safety Factor and Initial Safety Factor (Reactivated Slow-moving Landslide Countermeasures)

The current safety factor is set considering factors such as the presence or continuity of landslide movement. The initial safety factor is set considering changes in natural conditions such as terrain before the implementation of countermeasures.

【Explanation】

1) Classification and Display of Safety Factors

The safety factor for reactivated slow-moving landslides, which changes with variations in triggering factors and the progress of construction plans, is classified and displayed as follows.

Current Safety Factor F_s: Broadly, the safety factor fluctuating due to changes in inducing factors is termed the current safety factor. However, when evaluating the sliding condition of a slow-moving landslide or performing back-calculation of soil strength parameters, the minimum safety factor during the evaluation period is specifically referred to as the current safety factor. This is typically classified by the landslide prevention work construction period, indicating the safety factor against the maximum water pressure at that time.

Critical Safety Factor F_c: The safety factor at the onset of slow-moving landslide movement (critical state), corresponding to $F_s = 1.0$.

Initial Safety Factor F_0: The safety factor at the starting point of the slow-moving landslide prevention works plan. Generally, it matches the current safety factor before landslide prevention works construction. However, if the initial safety factor is defined as the point when terrain modification or similar changes are made from the current state, it differs from the current safety factor.

Planned Safety Factor F_p: The safety factor to be achieved by the entire slow-moving landslide prevention works.

2) Setting the Current Safety Factor

The only safety factor F_s that humans can truly evaluate as correct is when the slow-moving landslide begins to slide, i.e., $F_s = 1.0$. While it is possible to distinguish between a moving slow-moving landslide state ($F_s < 1.0$) and a stationary state ($F_s \geq 1.0$), the true safety factor value for each state cannot be known. If sufficient data on

displacement before and after slow-moving landslide movement and pore water pressure observations are available, the safety factor at the critical state ($F_s = F_c = 1.0$) can be used as the current safety factor at the point of maximum pore water pressure. Since direct measurement of pore water pressure on the slip surface is difficult, in Japan, pore water pressure is generally estimated from observed groundwater levels.

However, if sufficient observation data is unavailable at the start of a slow-moving landslide project, this analysis cannot be performed. In such cases, Japan empirically adopts the following standard values for the current safety factor.

＜Standard Safety Factor Values in Japan＞

1) When continuously active
F_s=0.95

2) When intermittently active due to rainfall, etc.
F_s＝0.98

3) When activity has subsided
F_s＝1.00

Once sufficient observation data on displacement and groundwater levels are available, and critical groundwater level observation data are obtained, it is desirable to review the current safety factor and soil strength parameters, and correct them to achieve a more appropriate current safety factor.

By increasing such analysis cases, it will become possible to establish standard values for the current safety factor in countries other than Japan.

3) Setting the Initial Safety Factor

The initial safety factor, which serves as the starting point for slow-moving landslide prevention work planning, typically provides the current safety factor value prior to countermeasure construction. If there are plans to alter the existing terrain due to reasons such as new road construction or widening, the safety factor after terrain modification may be calculated using stability analysis formulas and adopted as the initial safety factor.

3. 3. 3 Setting Soil Parameters (Countermeasures for Reactivated Slow-moving Landslides)

Soil parameters used for slope stability analysis shall be determined based on an evaluation of the target slope or slow-moving landslide's movement conditions and the results of soil tests on the sliding surface clay.

【Explanation】

1) Setting Soil Strength Parameters

Generally, shear strength parameters on the slip surface of the same reactivated slow-moving landslide mass also vary by location. Furthermore, since slope stability analysis equations based on the limit equilibrium method cannot calculate the true safety factor, the safety factor obtained from forward analysis using soil strength parameters directly derived from soil tests often lacks consistency with the dynamic behavior of reactivated slow-moving landslides. This lack of consistency makes it difficult to evaluate the effectiveness of subsequent countermeasures, significantly reducing the reliability of the safety factor achieved by those measures. For example, even if the calculated safety factor exceeds the design safety factor, it becomes difficult to determine whether the reactivated slow-moving landslide has truly achieved the design safety factor. Therefore, soil strength parameters are set using inverse analysis to ensure the calculated safety factor aligns with the dynamic conditions.

Depending on how c' and φ' are set, the following methods exist for inverse analysis.

a) Method for calculating *c'* by setting *φ'*

This method involves determining *φ'* using various residual strength tests (ring shear tests, repeated single-plane shear tests, etc.) on undisturbed or disturbed samples of the collected slip surface clay, then substituting this value into the stability analysis formula to calculate *c'*.

The calculated *c'* value includes analytical errors inherent in two-dimensional analysis and the limit equilibrium method itself, as well as the influence of resistance forces at the sidewalls and end sections. Consequently, it often yields values larger than the actual value. However, when estimating the effectiveness of prevention structures, the impact of overestimating *c'* is smaller compared to the impact of overestimating *φ'* (details are provided in the reference).

b) Method for calculating *φ'* by setting *c'*

This method involves determining *c'* from soil test results of the collected slip-face clay and substituting it into the stability analysis formula to calculate *φ'*.

The *φ'* value calculated here often includes analytical errors inherent to two-dimensional analysis and the limit equilibrium method itself, as well as effects such as resistance at the sidewall and terminal sections, resulting in values that are frequently larger than the actual value. Consequently, there is a risk of overestimating the effectiveness of slow-moving landslide prevention works. Therefore, it is important to verify the obtained *φ'* value by referring to soil test results and literature information (details are provided in the reference).

2) Unit Weight of the Mobile Layer

When high-quality boring cores are available, the most appropriate method is to measure the total weight of the core box containing the mobile layer portion and determine the average unit weight of the mobile layer. If exploratory boring has not been conducted, sampling via trench investigation or auger boring should be performed to determine the unit weight. If such investigations are not feasible, the unit weight must be estimated based on

the geological conditions of the target slope. In Japan, a method setting the general unit weight of the mobile layer, γ_t, to 18 kN/m³ has been adopted for about 30 years. The Road Earthwork - Temporary Structures Engineering Guidelines (JRA, 1999) presents the unit weight of soil as shown in Table 3.3.1. Within this table, γ_t = 18 kN/m³ is considered to have been the standard value used for dense cohesive soil.

Table 3.3.1 Moist Unit Weight of Soil (JRA, 1999)

Soil Type	Dense (kN/m³)	Loose (kN/m³)
Gravelly Soil	20	18
Sandy Soil	19	17
Cohesive Soil	18	14

References

JRA (1999): *Guidelines for Highway Earthwork: Temporary Structures*, Japan Road Association (JRA), pp.29.

【Reference Notes】

1) Method for Determining Cohesion *c*' from Thickness of Mobile Layer

In Japan, the method of setting cohesion *c*' according to the vertical thickness of slow-moving landslide soil blocks, as shown in the table below, is widely used. Another method exists that uses the average vertical thickness instead of the maximum value to set cohesion c' in the same manner.

Maximum vertical layer thickness *h* of slow-moving landslide mass (m)	Cohesion *c*' (kN/m²)
5	5
10	10
15	15
20	20
25	25

*For h > 25 m, c' = 25 kN/m²

This method targets reactivated slow-moving landslides in Japan's slow-moving landslide countermeasure projects. It applies to reactivated slow-moving landslides where the slip surface strength is known to reach the residual strength, and the cohesion c is approximately zero ($c \approx 0$). Adopting $c \approx 0$ in such re-sliding landslides causes various issues (e.g., calculating the effectiveness of slow-moving landslide countermeasures on dam impoundment slopes or the effectiveness of earth-removal works). This method was developed to artificially set $c > 0$. This method originated in an era when techniques for collecting slope clay samples were limited, and ring shear tests to determine residual strength of slope clay were not widely adopted. Ideally, the internal friction angle φ for residual strength should be determined via ring shear tests or similar methods, and cohesion c should be calculated inversely.

2) Impact of Overestimating Soil Strength Parameters (*c*', *φ*') on Evaluating Countermeasure Effectiveness

Overestimating the internal friction angle φ' leads to an overestimation of the effectiveness of ground anchor shear-resistance increasing effect and groundwater drainage works. This results in an erroneous assessment that the planned safety factor can be achieved with fewer countermeasures, leading to a design on the unsafe side.

Overestimating cohesion c' leads to an overestimation of the effectiveness of earth removal works. Therefore,

caution is required when earth removal works are the primary countermeasure.

3. 3. 4 Setting the Design Safety Factor (Measures for reactivated slow-moving landslides)

The design safety factor is a target value for enhancing slope stability and ensuring slope preservation through slow-moving landslide prevention works. Its determination shall be based on a comprehensive assessment of the slow-moving landslide phenomenon and scale, the importance of the preservation target, and the anticipated extent of damage caused by the landslide.

【Explanation】

The design safety factor is the benchmark value for determining the scale of slow-moving landslide prevention works (type and quantity of countermeasures, and their layout). It must be set according to the scale of the slow-moving landslide and the importance of the protected assets. Even if the protected assets are critical, setting a large design safety factor for a large-scale slow-moving landslide can result in an enormous quantity of countermeasures required to achieve it. This, in turn, leads to enormous costs, potentially making the countermeasure project unfeasible.

Taking these factors into account, the general guidelines for design safety factors in Japan are as follows:

1) Those with significant impact on numerous lives, houses, roads, railways, rivers, and other public facilities
   Design Safety Factor (F_p) = 1.10 to 1.20
2) Those of particularly large scale with considerable impact on houses and public facilities, or those requiring immediate safety measures as emergency countermeasures
   Design Safety Factor (F_p) = 1.05 to 1.10

The safety factors described here indicate the degree of improvement in the safety factor achieved through prevention works, assuming the current safety factor is F_s = 0.95 to 1.00 based on the slow-moving landslide's movement status. They do not indicate the slope's stability after construction.

Additionally, there are cases where a design safety factor F_p = 1.5 has been adopted when particularly important assets requiring protection are present.

【Reference Notes】

As of January 2024, publicly available data indicates there are 7,957 designated slow-moving landslide prevention zones in Japan (MLIT,2023; MAFF,2023; Yamanashi Prefecture, 2021; Shiga Prefecture, 2024; Kagawa Prefecture, 2024). These zones are the primary targets for slow-moving landslide countermeasure projects. Numerous slow-moving landslide countermeasure projects also occur in areas not designated as slow-moving landslide prevention zones, such as slow-moving landslide disasters on road cut slopes. The standard values for the planned safety factor mentioned above were established based on experience accumulated from numerous slow-moving landslide countermeasure projects conducted in Japan since the enactment of the slow-moving Landslide Prevention Act in 1958. When applying these standards to countries other than Japan, it is advisable to initially adopt the same standard values as Japan and then review them if any issues arise.

In countries where forward analysis is the standard, a design safety factor F_p=1.5 is often adopted. This is likely because soil samples tested in soil tests are frequently not from the most vulnerable geological conditions within the target ground, but rather from samples of relatively good quality where sampling is feasible. This accounts for

the risk that forward analysis may yield a safety factor larger than the actual value. In Europe and America, comprehensive countermeasure projects are often not implemented for large-scale slow-moving landslides. Therefore, setting the design safety factor at F_p=1.5 is appropriate for collapses and relatively small slow-moving landslides where realistic countermeasures are feasible even with a large safety factor. Implementing countermeasures corresponding to a design safety factor F_p=1.5 is often difficult for large-scale reactivated slow-moving landslides. Furthermore, global design software in Europe and the US that supports forward analysis often incorporates reliability design approaches, such as using slope failure probability based on probabilistic evaluation of soil parameters.

References

MLIT (2023): National Digital Geographic Information— Distribution of Slow-moving Landslide Prevention Areas, Ministry of Land, Infrastructure, Transport and Tourism (MLIT).

MAFF (2023): Summary Table of Slow-moving Landslide Prevention Areas Under the Jurisdiction of the Rural Development Bureau, Ministry of Agriculture, Forestry and Fisheries (MAFF), （https://www.maff.go.jp/j/nousin/noukan/tyotei/t_zisuberi/attach/pdf/index-4.pdf, Accessed 15 Jan. 2024）

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Kagawa Pref. (2024): Open Data KAGAWA, （https://opendata.pref.kagawa.lg.jp/dataset/, Accessed15 Jan. 2024）.

3. 4 Examples of Descriptions for Other Technical Standards

3. 4. 1 Setting Pore Water Pressure (Measures for Reactivated Slow-moving Landslides)

The pore water pressure used in stability analysis shall be appropriately set according to the purpose, considering the pore water pressure distribution throughout the entire target block.

【Explanation】

Stability analysis is performed according to the intended purpose, such as estimating soil strength parameters, evaluating the effectiveness of slow-moving landslide prevention works, and assessing post-construction landslide stability. Therefore, the pore water pressure used in stability analysis should be the value at an appropriate point in time within its time-varying range.

The pore water pressure used in stability analysis should be obtained from the groundwater zone near the slip surface, which correlates with slow-moving landslide movement. However, since measuring the pore water pressure acting on the slip surface is difficult, groundwater levels are often observed for convenience, and their head height is used as a substitute for pore water pressure. When using a full-bore strainer well for groundwater level observation, the observed level is influenced by multiple groundwater zones. Therefore, a method employing a partial-bore strainer well, where the upper and lower borehole walls are sealed to minimize the influence of other groundwater zones, is also adopted.

1. Pore Water Pressure in Estimating Soil Strength Parameters

It is desirable to use the pore water pressure at the critical state. If the critical state cannot be confirmed, use the pore water pressure determined from the maximum water level observed during the observation period.

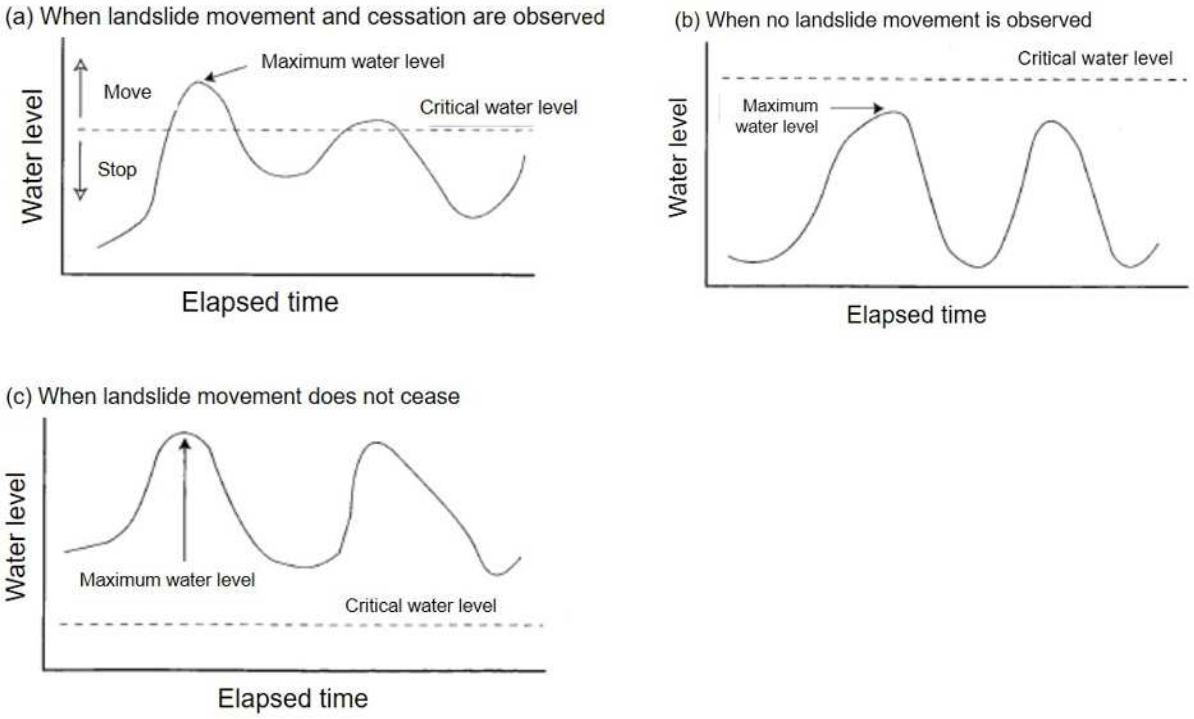


Figure 3.4.1 Relationship between Maximum Water Level and Critical Water Level

2. Pore Water Pressure Used for Determining Slow-moving Landslide Prevention Work Types, Methods, Scale, and Estimating Effectiveness

The pore water pressure used here should ideally correspond to the state where slow-moving landslides are most likely to occur (the most dangerous state), and typically corresponds to the highest observed water level. However, if the precipitation at the time the highest water level was observed in a groundwater level gauge installed after the slow-moving landslide occurred is significantly lower than at the time of the slow-moving landslide, the observed highest water level may not represent the most dangerous state, potentially preventing the formulation of an appropriate slow-moving landslide prevention works plan. Furthermore, the observed highest water level may sometimes be at a depth deeper than the slip surface. In such cases, the pore water pressure should be corrected as necessary.

3. Pore Water Pressure in Stability Assessment After Slow-moving Landslide Prevention Works

The pore water pressure here shall be set considering the highest observed water level, the highest water level corresponding to anticipated future torrential rainfall, and other factors.

4. Design of Cut Work

4. 1 Overview of Cut Work

Cut work is fundamental for road development in mountainous areas, but slope failures associated with cut work frequently occur. To properly design cut work, it is necessary to determine the slope gradient based on the soil type, geological conditions, and cut height of the natural ground, using information from various surveys and analyses, such as the appropriate cut slope gradient for the geology and understanding of the geological structure, to ensure stability. Note that earth-cutting work in residential land development faces similar issues to road development earth-cutting. Furthermore, earth-dumping work implemented as a slow-moving landslide countermeasure is also a type of earth-cutting work.

4. 2 Key Issues and Actions to Take in Internationalization

4. 2. 1 Differences with Other Countries Regarding Standard Cut Slope Grades and Standard Ground Classification for Cuts

(1) Issues

In Japan, while the fundamental approach is to evaluate the stability of each cut slope individually, empirical cut slope gradients are often adopted for sections where survey results indicate no particular problems.

Conversely, in Europe and Indonesia, standard cut slope gradients are not specified in codes or guidelines. Instead, stability assessments are conducted individually for problematic geology or long, steep cut slopes. Even if engineers recognize cut slope stability gradients as based on experience, some countries deliberately avoid setting standard cut slope gradients, while others lack the concept of standard cut slope gradients altogether.

When ultimately applying Japan's standard cut slope gradient, differences in climatic and geological conditions make direct application difficult. Furthermore, Japan's standard cut slope gradient range (1:0.5 to 1:1.2) is broad for certain materials like soft rock, and the lack of clear criteria for selecting which gradient within this range to adopt is also an issue.

Furthermore, Japan's standard cut slope gradient classification prioritizes constructability, which may not be universally understood. When applying Japanese standards, issues arise in determining how to apply Japan's rock mass classification to the target country's geology and how to adjust the standard cut slope gradient based on the target country's geological and climatic conditions.

(2) Explanation

1) Rock Mass Classification Based on Purpose and Application

Japan features a complex distribution of diverse geological zones, with steeply dipping strata due to folding and numerous faults. In contrast, stable continents typically have fewer faults and simpler geological structures. Thus, the geology of other countries often differs significantly from Japan's. Reviewing rock mass classifications used in earthwork projects abroad reveals numerous examples where rock mass is classified based on rock type and degree of weathering.

In Japan, while various rock mass classifications are used depending on the purpose and application (Table 4.2.1), for civil engineering construction (especially earthwork), classification is based on excavation difficulty and workability.

Note that the resulting classifications of soft rock and hard rock are not necessarily equivalent to the soft rock and hard rock determined by rock type in other countries.

Table 4.2.1 Various Classification Methods for Natural Ground in Japan Based on Purpose and Intended Usee

	Classification Method	Purpose/Use
Soil Classification	Engineering classification of ground materials (classification by the Japanese Geotechnical Society) (Japanese Unified Soil Classification)	Common understanding of strength
	Soil Classification for Construction Excavated Soil	Effective utilization, proper treatment
	Classification by Civil Engineering Construction Difficulty	Construction Difficulty Assessment and Unit Price Calculation, Cut Slope Gradient Selection
Rock Classification	Classification by Origin (Major Categories: Sedimentary Rocks, Igneous Rocks, Metamorphic Rocks)	Geological Map Creation, Common Understanding of Attributes Based on Geological Classification
	Classification by Geological Era of Formation (Paleozoic, Tertiary, etc.)	Geological Map Creation, Common Understanding of Attributes Based on Geological Classification
	Classification by Distribution Location and Common Names (Chichibu Paleozoic Formation, Silica Sand, etc.)	Geological Map Creation, Shared Understanding of Attributes Based on Geological Classification
	Classification by Civil Engineering Construction Difficulty (Hard Rock, Soft Rock)	Construction Difficulty Assessment and Unit Price Calculation, Selection of Cut Slope Gradient

2) Classification of Natural Ground and Standard Cut Slope Grades in Japanese Slope Stabilization

Table 4.2.2 shows the classification of natural ground and standard cut slope grades used in Japanese slope stabilization. Standard cut slope grades are set based on the type and condition of the slope materials, as well as the height of the cut.

Even within a single materials classification, the standard cut slope gradient has a range of values. The specific value within this range to be adopted is determined based on the results of topographical and geological surveys. Note that the *Design Guidelines for Highways* (MLIT, 2020), while conforming to the *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization* (JRA, 2009b), further subdivides the bedrock classification for determining cut slope gradients. Hard rock is subdivided into “Hard Rock” and “Medium Hard Rock,” while soft rock is subdivided into “Soft Rock” and “Weathered Rock.” Standard cut slope gradients corresponding to each classification are specified.

Table 4. 2.2 Classification of Natural Ground and Standard Cut Slope Grades in Japanese Slope Stabilization Measures (Example with Subdivided Rock Categories)
(Based on *Design Guidelines for Highways* (MLIT, 2020), with some additions)

Ground Classification		Cut Height	Slope (V:H) (JRA, 2009b)	Standard Slope Value (V:H)
Hard Rock※			1:0.3 -1:0.8	Hard Rock 1:0.3 Medium Hard Rock 1:0.5
Soft Rock※			1:0.5 - 1:1.2	Soft Rock 1:0.5 - 1:0.7 Weathered Rock 1:0.7 - 1:1.2
Sand	Non-compact, poorly distributed grain size		1:1.5 -	1:1.8
Sandy Soil	Compacted material	5m or less	1:0.8 - 1:1.0	1:1.0
		5 - 10m	1:1.0 - 1:1.2	
	Loose material	5m or less	1:1.0 - 1:1.2	1:1.2
		5 - 10m	1:1.2 - 1:1.5	
Sandy soil mixed with gravel or boulders	Compact, or with good particle size distribution	10m or less	1:0.8 - 1:1.0	1:1.0
		10 - 15m	1:1.0 - 1:1.2	
	Loose material or material with poor grain size distribution	10m or less	1:1.0 - 1:1.2	1:1.2
		10 - 15m	1:1.2 - 1:1.5	
Cohesive soil		10m or less	1:0.8 - 1:1.2	1:1.2
Clayey soil containing boulders or cobbles		5m or less	1:1.0 - 1:1.2	1:1.2
		5 - 10m	1:1.2 - 1:1.5	

(Note) 1. Silt is included in cohesive soils. Other soil types shall be considered separately.
※: The classification of hard rock and soft rock is based on the difficulty of earthwork, not rock type, according to a classification unique to Japan.

3) Japan's meteorological and geological conditions

Japan's overall average annual precipitation is approximately 1,800 mm, but there are significant regional variations, ranging widely from a maximum of about 4,600 mm to about 800 mm (Figure 4.2.1). Furthermore, during torrential rainfall events, precipitation exceeding 1,000 mm over three days frequently occurs.

Meanwhile, as shown in Figure 4.2.2, oceanic plates around Japan move toward continental plates at a rate of several centimeters per year, subducting beneath them. The complex forces exerted by multiple plates in this region make it one of the world's most seismically active zones (Japan and its surrounding areas account for about one-tenth of the world's earthquakes (Table 4.2.3)).

Japan's geology has developed through accretionary processes associated with oceanic plate subduction (Figure 4.2.4). Consequently, accretionary complexes from multiple geological eras have accumulated over time, resulting in the distribution of diverse geological formations spanning a wide range of ages (Figure 4.2.5). Consequently, the geological structure is complex, geological conditions vary significantly, and even within the same slope, geological conditions can change drastically. Furthermore, volcanic activity has been active throughout geological time, and faults and active faults are numerous throughout the country (Figure 4.2.3). The widespread presence of volcanic pyroclastic material prone to alteration and clayification, combined with fracturing due to tectonics and metamorphism from pressure and heat, results in fragile geology. The convergence of these conditions creates an

environment susceptible to weathering, making it prone to failures such as landslides.

This is a factor contributing to the wide range of standard cut slope gradients. Therefore, when considering standard cut slope gradients in target countries, differences in geology, climate, and other factors compared to Japan must be taken into account.

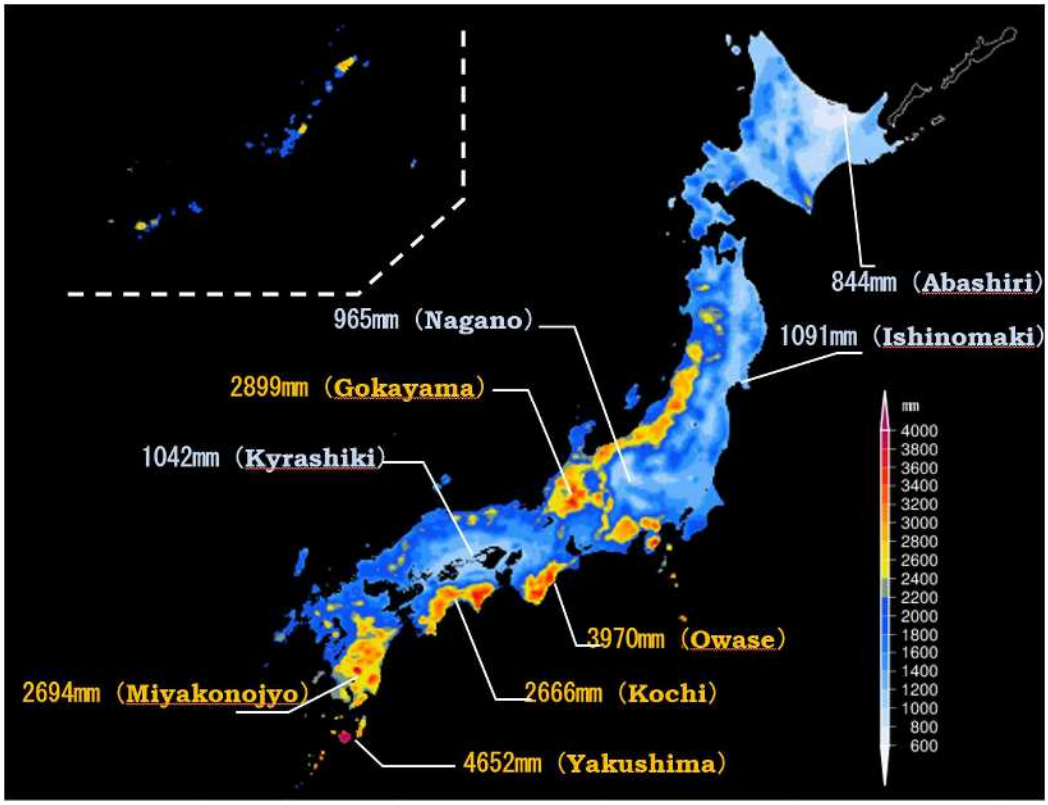


Figure 4.2.1 Map of Japan's Annual Average Precipitation Distribution

(Map of Annual Precipitation from JMA(2014), https://www.data.jma.go.jp/obd/stats/etrn/view/atlas.html)

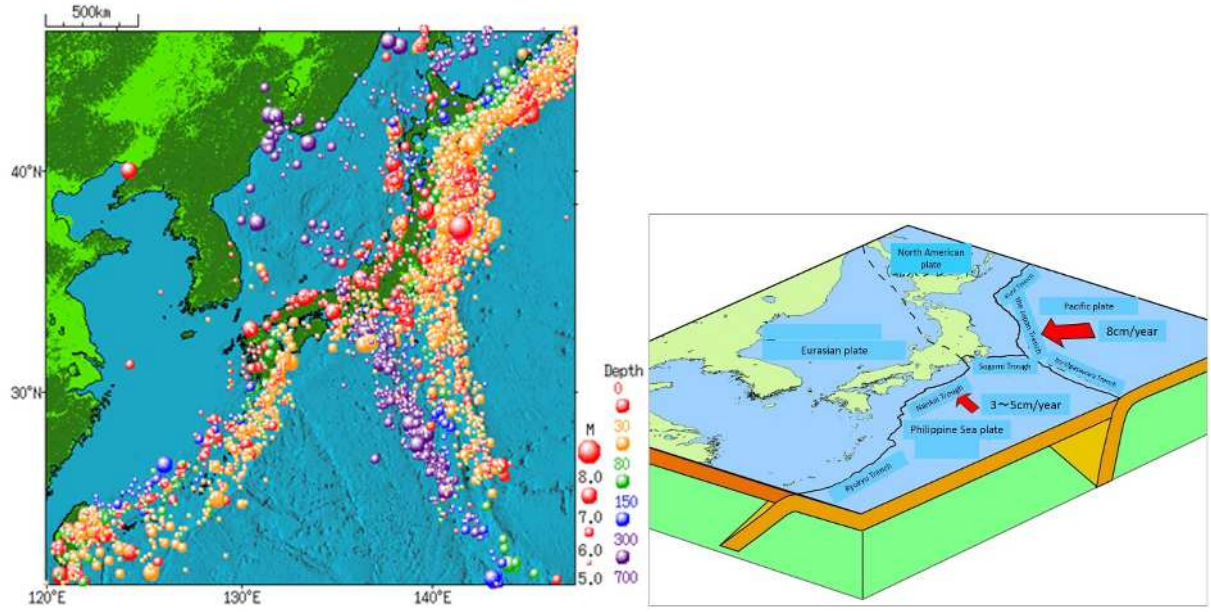


Figure 4.2.2 Distribution of Earthquakes Occurring Near Japan (1960–2011) and Schematic Diagram of Plates Near Japan

(JMA(2019), https://www.data.jma.go.jp/svd/eqev/data/jishin/about_eq.html)

Table 4.2.3 Number of Earthquakes in Japan and Surrounding Areas vs. Global Earthquake Frequency (Annual Average)

(Compiled from data from JMA(2021), https://www.jma.go.jp/jma/kishou/know/faq/faq7.html)

	Number of Earthquakes in Japan and Surrounding Areas (Annual Average)	Global Earthquake Count (Annual Average)	Percentage of Global Earthquakes Occurring in Japan and Surrounding Areas
M8.0 or Greater	0.2 (2 per decade)	1	20%
M7.0 - 7.9	3	17	18%
M6.0 - 6.9	17	134	13%
M5.0 - 5.9	140	1,319	11%
M4.0 - 4.9	Approx. 900	13,000	7%
M3.0 - 3.9	Approx. 3,800	130,000	3%

※1 Number of earthquakes in Japan and surrounding areas: Japan Meteorological Agency hypocenter data from 2001 to 2010

※2 Number of earthquakes worldwide: USGS (United States Geological Survey) data since 1900

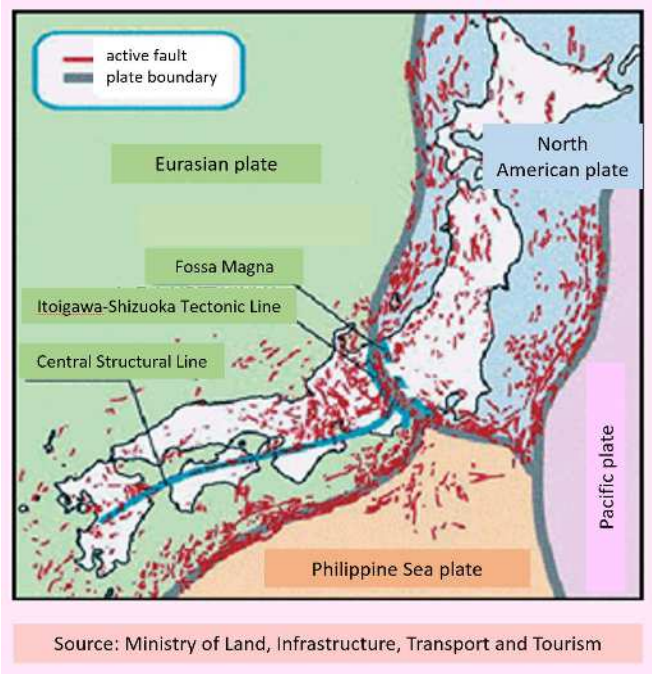


Figure 4.2.3 Distribution of Plates and Active Faults around Japan
(2005 White Paper on National Land and Transport(MLIT, 2006):
https://www.mlit.go.jp/hakusyo/mlit/h17/index.html)

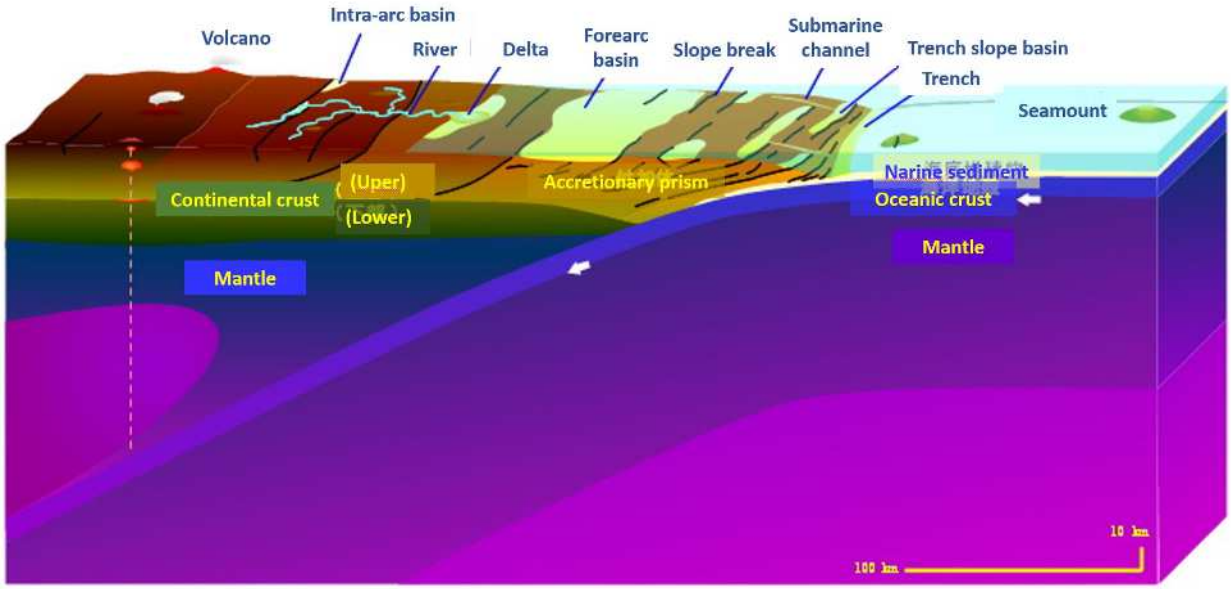


Figure 4.2.4 Schematic Cross-Section of the Subduction Zone
(GSJ(2024): https://www.gsj.jp/en/education/geomap-e/subduction-e.html)

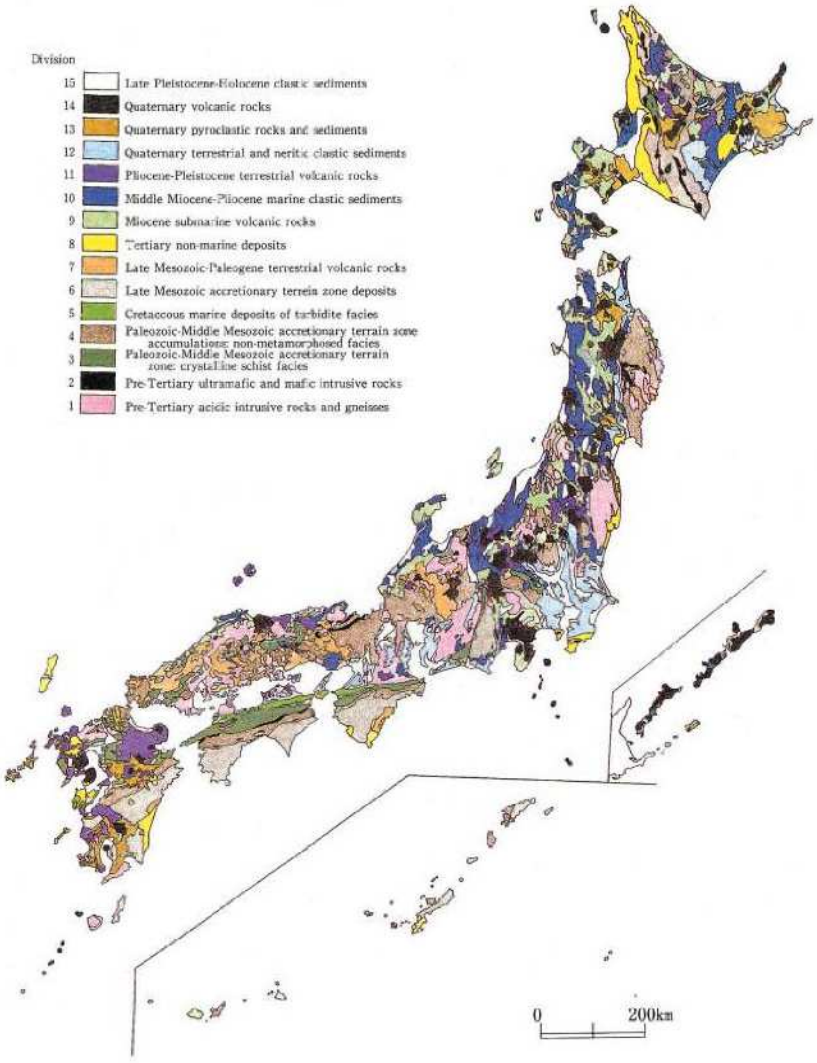


Figure 4. 2.5 Geological Distribution in Japan （JLS, 2002）

4) Criteria for Classifying Ground Conditions in Japanese Slope Stabilization Measures

In Japan, for earthwork such as cut slopes, indicators like the operability of machinery (heavy equipment) and elastic wave velocity are used to comprehensively evaluate diverse ground conditions. Rock primarily requiring blasting is classified as hard rock, rock primarily requiring ripper work is classified as soft rock, and material directly excavable by bulldozer is classified as earth and sand (e.g., JSSMFE (1974), Adachi (1984)).

The *Guidelines for Highway Earthwork: Soil Investigation* (JRA, 1986) set the criteria for determining sand/soil vs. soft rock based on the feasibility of excavation using the dozer blade of a 21-ton class bulldozer, and the criterion for distinguishing soft rock from hard rock is the feasibility of ripper work using a 32-ton class bulldozer. This indicates a correlation between the application limits of heavy machinery used for excavation and the in-situ elastic wave velocity (*Outline for Highway Earthwork* (JRA, 2009)) (Table 4.2.4). The ground encountered in earthwork ranges from hard rock to soft clay, exhibiting extreme diversity. Since the distributed geology varies by country and region, Japan's ground classification (Table 4.2.4), which considers workability with construction

machinery, offers high versatility.

Table 4. 2.4 Guidelines for Classifying Ground Based on Workability with Heavy Machinery and Ground Elastic Wave Velocity
(Compiled from JRA (1986) and JRA (2009a))

Ground Classification	Criteria based on construction feasibility with heavy machinery, JRA (1986)	Ground elastic wave velocity, JRA (2009a)
Soil	Excavation possible using a 21-ton class bulldozer's dozer blade	Approximately 700 m/s
Soft Rock	Excavation using a 21-ton class bulldozer's dozer blade is difficult Ripper work possible using a 32-ton class bulldozer	Approximately 2.0 km/s or less
Hard Rock	Ripper work using a 32-ton class bulldozer is difficult	Approximately 2.0 km/s or more

Table 4.2.2 classifies soils based on this into “soft rock,” “hard rock,” and “soil/sand.” “Soil/sand” is further subdivided based on its properties into sand, sandy soil, gravel or rock-mixed sandy soil, cohesive soil, and rock or boulder-mixed cohesive soil.

Note that sandy soil is divided into “dense” and “non-dense” types; sandy soil with an N-value greater than 20 to 30 is judged to be “dense.”

5) History of Standard Cut Slope Grades in Japanese Slope Stabilization

Standard cut slope grades in Japanese slope stabilization were steeper in the 1950s. However, subsequent analyses of failure and non-failure cases gradually expanded the overall range of acceptable grades toward gentler slopes. Figure 4.2.6 below presents research findings that complement the cut slope gradients in Table 4.2.2.

Figure 4.2.6 shows the relationship between the elastic wave velocity measured at the cut location and the actual cut slope gradient constructed at that location. The slopes circled in Figure 4.2.6 are those that failed at that gradient, distributed in the lower left part of the figure. On the other hand, sound cut slopes are mainly distributed in the upper right part of the figure.

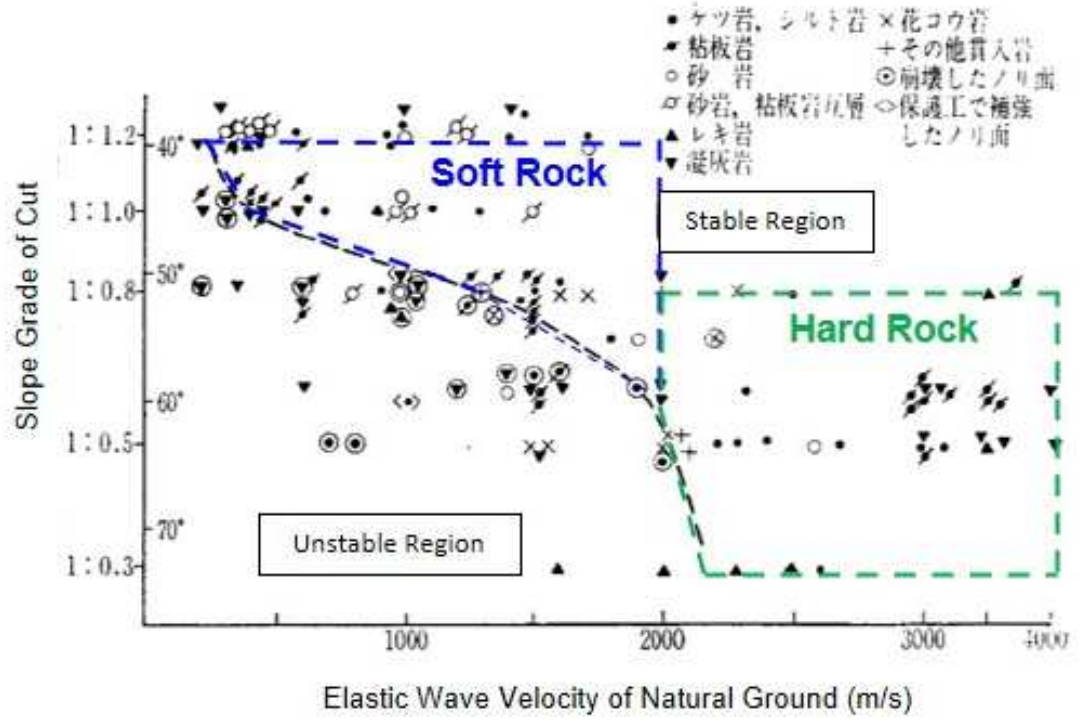


Figure 4.2.6 Boundary between stable and unstable zones derived from in-situ rock elastic wave velocity and cut slope gradient (Added to Okuzono (1972))

Table 4.2.4 shows the boundary between soft rock and hard rock defined by an elastic wave velocity of 2,000 m/s. In Figure 4.2.6, soft rock and hard rock are distinguished, and their stable regions are organized as follows: for soft rock, the slope ratio ranges from 1:0.5 to 1:1.2 (blue dashed line in the figure); for hard rock, the slope ratio ranges from 1:0.3 to 1:0.8 (green dashed line in the figure). This classification result is identical to the standard cut slope gradients for hard rock and soft rock in Table 4.2.2, thereby supporting the validity of the standard cut slope gradients for soft rock and hard rock in Table 4.2.2. Thus, since Japan verifies the validity of Table 4.2.4 in Figure 4.2.6, when applying the Japanese ground classification system to other countries, it is desirable to conduct similar analyses in the target country to verify the validity of the newly established standard cut slope gradients for that country, which will naturally differ from those in Japan.

6) Addressing the Wide Range of Standard Cut Slope Grades in Japan

The standard cut slope grades for slope stabilization in Japan (Table 4.2.2) are 1:0.3 to 1:0.8 for “hard rock” and 1:0.5 to 1:1.2 for “soft rock,” which is a broader range compared to the cut slope grades for earth slopes.

In Japan, a rock classification system dividing bedrock into six categories (A, B, CH, CM, CL, D) has been proposed. The relationship between this rock classification and the “hard rock” and “soft rock” ground classifications has been examined using elastic wave velocities. Based on this, when the standard cut slopes for “hard rock” and “soft rock” are subdivided, they are organized as shown in Table 4.2.5. Note that Table 4.2.5 is strictly for reference purposes and does not constitute a Japanese standard. The current practice in Japan is to determine the cut slope gradient for a site by referring to the existing gradient of the natural slope and the cut slope gradient in nearby areas with the same geology.

Table 4.2.5 Results of Grading for Cut Slopes When Subdividing the "Hard Rock" and "Soft Rock" Categories (Reference)

Rock Grade Classification			Cut Slope Gradient for Subdivision (Organized Results)	Elastic Wave Velocity (km/s)	Cut Slope Gradient (V:H) per Figure 4.2.6	Standard Cut Slope Gradient in Japanese Slope Stabilization (Table 4.2.2)
Hard rock	Hard rock	A	-1:0.3 (Up to 15m height)	＞5.0	1:0.3 - 1:0.8	1:0.3 - 1:0.8
		B	1:0.3 - 1:0.5 (Up to 7m height)	3.5 - 5.0	1:0.3 - 1:0.8	
	Medium Hard Rock	C_H	1:0.5 - 1:0.8 (Up to 7m height)	2.0 - 4.0	1:0.3 - 1:0.8	
Soft rock	Soft rock II	C_M	1:0.8 - 1:1.0 (Up to 7m height)	1.0 - 3.0	1:0.5 - 1:0.9	1:0.5 - 1:1.2
	Soft Rock I	C_L	1:0.8 - 1:1.2 (Up to 5m height)	0.5 - 2.5	1:0.5 - 1:1.0	
		D	1:1.2 - (Up to 5m height)	0.3 - 1.0	1:0.9 - 1:1.2	

7) Differences between Standard Cut Slope Grades in Other Countries and Japan

Comparisons of rock mass classifications and standard cut slope grades between other countries and Japan are shown in Table 4.2.6 and Table 4.2.7. Since geological conditions, weather/climate conditions, and rock mass classification methods differ between other countries and Japan, the comparison focuses solely on the distribution range of standard cut slope grades. (Rock mass classifications and standard cut slope gradients for other countries are provided as reference material).

The distribution range of standard cut slope gradients for earth and rock tends to shift toward steeper gradients in the Central Asian regions of India and Bhutan compared to Japan. Conversely, the ranges are broader and more variable in the West Asian region (Tajikistan), Canada, and the United States. The African region shows a tendency toward slightly gentler gradients.

Meanwhile, for rock, the distribution range of standard cut slopes shows a tendency toward steeper slopes in the Central Asian regions of India and Bhutan compared to Japan. In contrast, Tajikistan in West Asia, Canada, and the American regions of the United States exhibit a wider range and greater variation. The African region shows a tendency toward slightly gentler slopes.

8) Differences in the Approach to Standard Cut Slope Grades Between Europe and Japan

In European countries such as France, Italy, and Switzerland, there are few geologic conditions where cut slope grades become problematic in mountainous areas. Furthermore, new road development is limited, with widening and upgrading to higher standards being the mainstream approach. In contrast, Japan has extensive new road development, including expressways, and faces an environment prone to slope failures associated with cut slopes. This necessitated establishing clear standard values for cut slope gradients.

Even in Europe, where stability assessments are performed for all cut slopes, the final determination of a cut slope gradient cannot be made without historical empirical values for cut slope gradients. Therefore, in practice, standard cut slope gradients are effectively utilized.

Table 4.2.6 Comparison of Standard Cut Slope Grades in Japan and Other Countries (Earth Slopes)

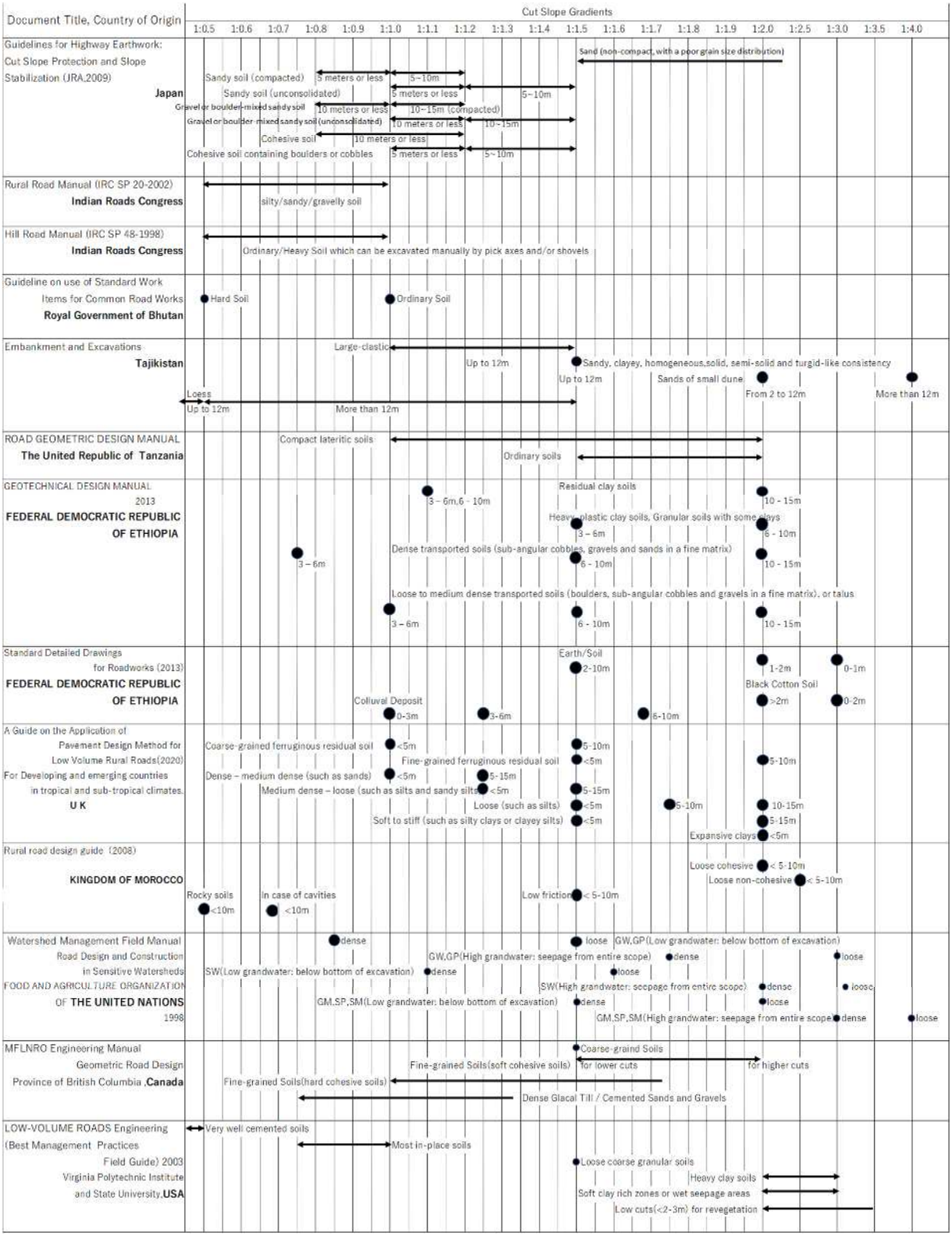
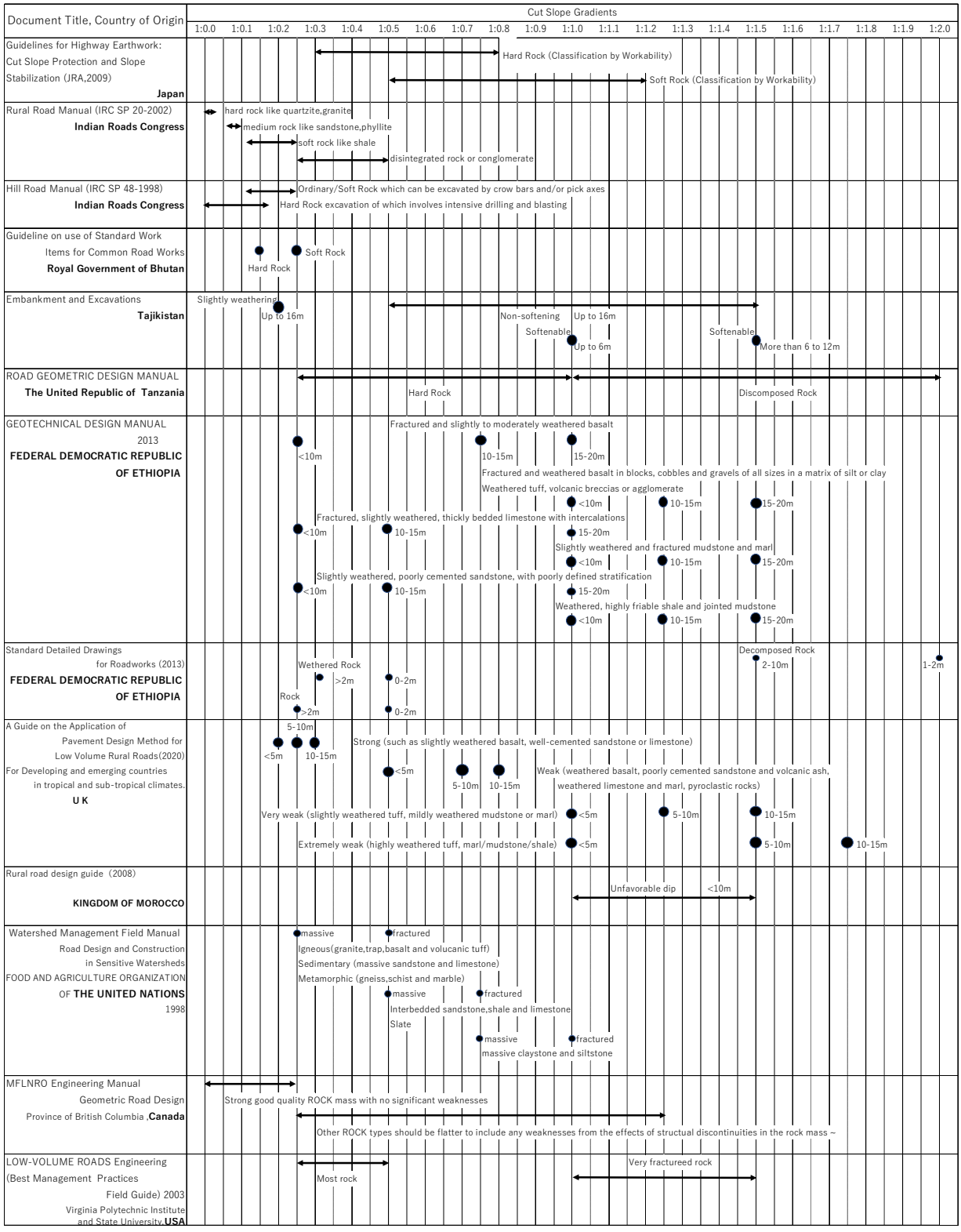


Table 4.2.7 Comparison of Standard Cut Slope Grades in Japan and Other Countries (Rock Slopes)



(3) Actions to Take

In the design of cut slopes, the basic approach is to conduct a detailed investigation and then evaluate the stability of each individual cut slope using stability analysis and other methods. However, even in these individual stability evaluations, past experience regarding the setting of cut slope gradients is referenced, making the accumulation of past experience values crucial. Based on this, the following response method is adopted.

If the target country's standard cut slope gradient criteria and their operational status are investigated and found to be acceptable, the existing standard cut slope gradient shall be utilized.

When no standards exist in the target country or operational issues arise, the rock mass classification used to determine the standard cut slope gradient for cut slopes should not be based on geological classification. Instead, it should follow the rock mass classification adopted in Japan, which considers constructability and elastic wave velocity. This involves determining which category (e.g., hard rock, soft rock) the target geology falls into. Subsequently, the standard cut slope gradient for each category should be set according to the specific conditions of the country. Furthermore, regarding the standard cut slope gradient for soft rock in Japan's rock mass classification, refer to Table 4.2.5 and consider subdividing the rock mass classification.

Furthermore, even when the rock classification is the same, standard cut slopes may differ by country or region due to variations in climatic conditions and other factors. Consequently, standard cut slopes different from those in Japan may be appropriate. For example, countries with less rainfall and fewer earthquakes than Japan may adopt steeper values within the standard cut slope gradient range, while countries with more rainfall and earthquakes than Japan may adopt gentler values within the same range. Furthermore, in countries with no earthquakes and low precipitation, slopes remain stable even with steeper gradients for the same rock mass classification, allowing for steeper standard slope gradients. Thus, the appropriate standard slope gradient for each rock mass classification category is determined based on construction experience and the standard slope gradients used in other countries (including neighboring countries) for similar geological conditions.

Furthermore, for cut slope cases in target countries, quantitative geological and soil properties (e.g., elastic wave velocity and water absorption rate) are determined through field measurements and laboratory tests. By organizing and analyzing these properties in relation to actual slope gradients for failure cases, and successful cases. By determining the safe slope gradient, this provides an objective indicator for evaluating the validity of the country's standard cut slope gradient and is considered useful for improving it. Therefore, actively implementing such objective evaluations is desirable.

Even in Europe, where no standard cut slope gradient is established, an empirical guideline for cut slope gradients is necessary. Furthermore, while the cut slope gradient at the implementation site is a major factor in considering cut stability, numerous other elements affecting cut stability exist, such as the geological structure of the site and the development of cracks. Therefore, a specific cut slope gradient must be set based on a comprehensive assessment of these factors. Additionally, it is necessary to examine the stability by considering disaster scenarios that could occur on that cut slope.

(4) Reference Materials

As reference materials, the following lists the rock mass classifications and standard slope gradients used in various countries.

(a) India

◆IS Soil Classification System (Indian Highway Conference)

This classification system was finalized by the Soil Engineering Sectional Committee and adopted by the Indian Standards Institution in 1970. It was developed by simplifying the U.S. Unified Soil Classification System (USCS) to enhance its versatility and was adopted as the IS system (Table 4.2.8). This classification focuses on the properties of soil.

Table 4. 2.8 IS Soil Classification System (Technical Handbook - Roads, RDPR, 2013)

Soil Group Description	Symbol	Unit Dry Weight, gm/cm³	CBR %
Well graded gravels and gravel sand mixture (fines < 5%)	GW	2.00 - 2.24	60 - 90
Poorly graded gravel and gravel sand mixture (fines < 5%)	GP	1.76 - 2.24	25 - 60
Silty gravel and gravel sand mixture (fines > 12%)	GM	2.08 - 2.22	20 - 80
Clayey gravels and gravel sand silt mixtures (fines < 5%)	GC	1.92 - 2.24	20 - 80
Well graded sand and gravely (fines < 5%)	SW	1.76 - 2.08	20 - 60
Poorly graded sands and gravely sand (fines < 5%)	SP	1.59 - 1.92	10 - 30
Silty sand and sand-silt mixtures (fines > 12%)	SM	1.92 - 2.16	10 - 40
Clayey sands and sand-clay mixtures (fines >12%)	SC	1.68 - 2.08	15 - 50
Inorganic silt, very fine rock floor	ML	1.60 - 2.00	5 -20
Clayey silt or fine sand, Inorganic clay, gravelly, sandy or silts	CL	1.60 - 2.00	5 - 150
Organic silt and silty clays	OL	1.44 - 1.60	3 - 8
Inorganic silt elastic and micaceous silts	MH	1.28 - 1.60	3 - 8
Inorganic fat clays	CH	1.44 - 1.76	3 - 5
Organic silt and clays	OH	1.28 - 1.68	2 - 4

Table 4. 2.1 Symbols adopted as per Indian Soil Classification System

First Alphabets		Second Alphabets	
G	Gravel	W	Well graded
S	Sand	P	Poorly graded
M	Slit	L	Low plasticity
C	Clay	I	Intermediate plasticity
O	Organic soil	H	High plasticity

This IS system classifies soils based on their properties, and it is the simplest, most straightforward, and easiest to use. As described later, some countries use it to classify soils for determining standard cut slope gradients (treating it as a classification linked to constructability).

◆*Hill Road Manual* (IRC SP 48-1998) (Indian Road Congress)

The Indian Road Congress's road design standards are specified by road type. Table 4.2.10 shows the *Hill Road Manual* (IRC SP 48-1998) (IRC,1998) for mountain roads. The classification of natural ground for determining the standard cut slope gradient is divided into soil and rock, similar to Japan's classification based on construction workability (Table 4.2.10).

Table 4. 2.10 Hill Classification and Standard Cut Slope Grades for Earthwork
(*Hill Road Manual* (IRC SP 48-1998, IRC, 1998))

Categorie	Remarks	Slope (H:V)
Ordinary/Heavy Soil	This comprises of organic soil, clay, sand, moorum and stiff clay which can bi excavated manually by pick axes and/or shovels or with dozers/excavators with normal efforts. Soil mixed with boulders is also deemed to come under this category.	1:1 - 1/2:1
Ordinary/Soft Rock	This comprises of soft varieties of rocks such as lime stone, sand stone, laterite, conglomerate or other disintegrated rocks, which can be excavated by crow bars and/or pick axes without blasting or with casual blasting.	1/4:1 - 1/8:1
Hard Rock	This covers any hard rock, excavation of which involves intensive drilling, and blasting. This can stand vertical or even overhanging cut depending on the type/mass and dip of rock.	80° - 90°

◆*Rural Road Manual* (IRC SP 20-2002) (Indian Roads Congress)

Similarly, the *Rural Road Manual* (IRC SP 20-2002, IRC, 2002) on rural roads, published by the Indian Roads Congress, provides a classification of subsoil types for specifying cut slope gradients (Table 4.2.11). This classification is based on the origin (rock type) of the rock.

Table 4. 2.11 Ground Classification for Specifying Cut Slope Gradients on Rural Roads by the Indian Roads Council（*Rural Road Manual* (IRC SP 20-2002, IRC, 2002)
(Original: Table 3.6: Side Slope for Rural Roads (IRC SP : 20 - 2002, Page No. 34))

Condition	Slope (H:V)
Cutting in silty/sandy/gravelly soil	1:1 to 1/2:1
Cutting in disintegrated rock or conglomerate	1/2:1 to 1/4:1
Cutting in soft rock like shale	1/4:1 to 1/8:1
Cutting in medium rock like sandstone, phyllite	1/12:1 to 1/16:1
Cutting in hard rock like quartzite, granite	Near Vertical

(b) Federal Democratic Republic of Ethiopia

◆*Standard Detailed Drawings for Roadworks*

The *Standard Detailed Drawings for Roadworks* (ERA,2013a), created in 2013, include a standard road cross-section and an attached table listing embankment heights and slope gradients corresponding to soil classifications (Table 4.2.12, "BACK/CUT SLOPE").

Table 4.2.12 Standard Cut Slope Grades Published in *Standard Detailed Drawings for Roadworks* (ERA,2013a)

SLOPE RATIO TABLE		
MARERIAL TYPE	"Depth of Cut /Ht of Fill (H)" [m]	BACK / CUT SLOPE
EARTH/SOIL	0 － 1	3:1
	1 － 2	2:1
	2 －10	3:2
	＞10	BENCHED SLOPE
DECOMPOSED ROCK	0 － 1	3:1
	1 － 2	2:1
	2 －10	3:2
	＞10	BENCHED SLOPE
WEATHERED ROCK	0 － 2	1:2
	＞2	1:3
ROCK	0 － 2	1:2
	＞2	1:4
BLACK COTTON SOIL	0 － 2	1:3
	＞2	2:1
COLLUVIAL DEPOSIT	0 － 3	1:1
	3 － 6	5:1
	6 －10	5:1
	＞10	BENCHED SLOPE

◆Road Design Standards (*Geotechnical Design Manual*)

The Road Design Standards (*Geotechnical Design Manual*, ERA, 2013b), newly established in 2013 with international assistance, includes soil classifications (Table 4.2.13) and rock classifications (Table 4.2.14) for determining cut slope gradients and cut heights.

Soil classification is based on experience and covers general soils. Special soils found in Ethiopia (e.g., Black cotton soil) are addressed in separate sections.

Rock classification is based on origin (rock type) and covers geological conditions commonly found in Ethiopia. The explanatory notes for this table state: "Generally, a slope ratio of 1:0.25 is acceptable if the geological conditions affecting slope stability are favorable and there are no construction issues. However, stability may be compromised due to the relationship between joint direction and slope orientation, potentially requiring supplementary measures. In such cases, from an economic perspective, a slope ratio of 1:0.5 may be considered."

Table 4.2.13 Soil Classification in Ethiopia (*Geotechnical Design Manual*, ERA, 2013b)

(Original: Table 4-8 Soil cut slope ratios (H:V) for preliminary design purposes)

Type of Material	Cutslope total height (m)			Remark
	3 – 6	6 - 10	10 - 15	
Residual clay soils	1:1	1:1	2:1	Consider benching when the slope height is above 6 m. Vegetation cover is highly recommended.
Heavy, plastic clay soils	1.5:1	2:1	--- ---	Keeping the slope dry is extremely important.
Granular soils with some clays	1.5:1	2:1	--- ---	Keep a constant slope. Appropriate drainage and vegetation is necessary.
Dense transported soils (sub-angular cobbles, gravels and sands in a fine matrix)	0.75:1	1.5:1	2:1	Reduce the upper portion to 1:1 to limit gully formation or widening.
Loose to medium dense transported soils (boulders, sub-angular cobbles and gravels in a fine matrix), or talus	1:1	1.5:1	2:1	Cover the slope with grass and other suitable plants and keep the slope dry.

Note: Slope angles are indicative and require site-specific assessment.

Table 4.2.14 Rock Classification in Ethiopia (*Geotechnical Design Manual*, ERA, 2013b)

(Original: Table 4-12: Indicative maximum cut slope angles (H:V) for rock slopes (without discontinuity control))

Type of Material	Cut-slope height			Remark
	<10m	10-15m	15-20m	
Fractured and slightly to moderately weathered basalt	0.25:1	0.75:1	1.00:1	Remove hanging blocks before cutting. This minimizes the formation of overhangs
Fractured and weathered basalt in blocks, cobbles and gravels of all sizes in a matrix of silt or clay	1.00:1	1.25:1	1.50:1	
Weathered tuff, volcanic breccias or agglomerate	1.00:1	1.25:1	1.50:1	
Fractured, slightly weathered, thickly bedded limestone with intercalations	0.25:1	0.50:1	1.00:1	Apply controlled excavation; benches and drainage are needed at different heights.
Slightly weathered and fractured mudstone and marl	1.00:1	1.25:1	1.50:1	Remove weathered and unconsolidated portion at low angles. Drainage is important to keep the slope dry. Reinforcement works and erosion controlling measures may be needed.
Slightly weathered, poorly cemented sandstone, with poorly defined stratification	0.25:1	0.50:1	1.00:1	Controlled or trim blasting is recommended so as not to damage the rock behind the cut face.
Weathered, highly friable shale and jointed mudstone	1.00:1	1.50:1	2.00:1	Shale has a characteristic of being deformed when wet. Hence, drainage systems are needed to keep the slope dry. Benches may also be required.

(c) Classification by the U.S. Forest Service (USFS), as adopted by the Food and Agriculture Organization of the United Nations (FAO)

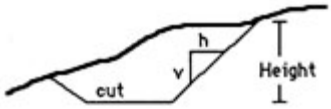
◆*Watershed Management Field Manual Road Design and Construction in Sensitive Watersheds* (FAO,1998)

This is a manual for road design in mountainous areas by the Food and Agriculture Organization of the United Nations (FAO).

Table 4.2.15 presents the soil classification system adopted by the FAO, based on the USFS (1973). Cut slope gradients are specified based on the soil classification system of the IS system. This classification focuses on soil properties to assess constructability. Rocks are classified according to their origin (rock type). Table 4.2.16 shows the FAO's classification of rock based on origin (rock type) for rock slopes, for which cut slope gradients are specified.

Table 4. 2.15 Soil Classification System Developed by the USFS and Adopted by the FAO (FAO,1998)

Table 18. Maximum cut slope ratio for coarse grained soils. (USFS, 1973).



Soil Type	Maximum Cut Slope Ratio (H:V)			
	Low groundwater (below bottom of excavation)		High groundwater [1] (see page from entire scope)	
	loose [2]	dense [3]	loose	dense
GW, GP	1.5 : 1	0.85 : 1	3 : 1	1.75 : 1
SW	1.6 : 1	1: 1	3.2 : 1	2 : 1
GM, SP, SM	2 : 1	1.5 : 1	4 : 1	3 : 1

[1] Based on material of saturated density approximately 19.6 kN/m³. Flatter slopes should be used for lower density material and steeper slopes can be used for higher density material. For every 5 % change in density, change the ratio by approximately 5%.
[2] Approximately 85% of maximum density.
[3] Approximately 100% of maximum density.

USFS(1973): Transportation Engineering Handbook, Slope design guide. Handbook No. 7709.11., USDA, Forest Service(USFS).

Note: For Soil Type: GW, GP—Low groundwater (below the bottom of the excavation)—dense, the value "85:1" is listed in the cited source, *Watershed Management Field Manual: Road Design and Construction in Sensitive Watersheds* (1973).

Table 4. 2.16 Maximum Cut Slope (Bedrock) According to the USFS, as Adopted by the FAO(FAO,1998)

Table 19. Maximum cut slope ratio for bedrock excavation (USFS, 1973).

Rock type	Maximum Cut Slope Ratio (H:V)	
	Massive	Fractured
Igneous (granite, trap, basalt, and volcanic tuff)	0.25.1	0.50:1
Sedimentary (massive sandstone and limestone;	0.25.1	0.50:1
interbedded sandstone, shale, and limestone;	0.50:1	0.75:1
massive claystone and siltstone)	0.75:1	
Metamorphic (gneiss, schist, and marble;	0.25.1	0.50:1
slate);	0.50:1	0.75:1
(serpentine)	Special investigation	

USFS(1973): Transportation Engineering Handbook, Slope design guide. Handbook No. 7709.11., USDA, Forest Service(USFS).

(d) British Columbia, Canada

◆MFLNRO(2018): *Engineering Manual - Geometric Road Design 2018*（Ministry of Forests, Lands, Natural Resource Operations and Rural Development: MFLNRO）

This is the slope gradient corresponding to the soil and rock classification in the Forest Road Design Manual of the Ministry of Forests, Lands, Natural Resource Operations and Rural Development (MFLNRO) in British Columbia, Canada. It is a classification specifying cut slopes with a height of less than 6 m (Table 4.2.17). This classification is based on experience and workability.

Table 4.2.17 Soil Classification by MFLNRO(MFLNRO,2018)

Table 3-4 General guidelines for cut and fill slope angles for use in forest road design

	Cut Slope	
	Examples of Material Types[a]	**Suggested Cut Slope Angles[b] for Cut Bank Height < 6 m[c]**
Coarse-grained Soils[d]	Road cuts in loose to compact SANDS or SANDS and GRAVELS (not cement and non-cohesive)	1½ H : 1 V
Fine-grained Soils	Road cuts in loose SILTS, or soft cohesive soils such as SILTY CLAYS or CLAYS (not consolidated and not cemented Road cuts in hard cohesive soils such as SILTY CLAYS or CLAYS (consolidated)	1½ H: 1 V for lower cuts to 2 H: 1 V for higher cuts 1 H : 1 V or flatter
Dense Glacial Till / Cemented Sands and Gravels	Road cuts in dense to very dense GLACIAL TILL (i.e., basal till), or cemented SANDS or SANDS and GRAVELS	¾ H : 1 V or flatter
Rock[f]	Road cuts in strong, good quality ROCK mass with no significant weaknesses	1/4 H : 1 V to vertical
	Road cuts in other ROCK types should be flatter to include any weaknesses from the effects of structural discontinuities in the rock mass, and other factors such as the strength of the rock material, and the spacing, aperture, roughness, filling, weathering, and orientation of discontinuities	1/4 H : 1 V to 1¼ H : 1 V

Notes:

a Not all material types in the soil groups are represented in the table.

b For the design of roads located in domestic watersheds, on sensitive terrain, or in other areas where transport of sediment or landslides may adversely affect resources, it may be necessary to use flatter cut and fill slope angles to reduce the hazard of erosion or failure of cut and fill slopes. During construction of roads, it may be necessary to build flatter slopes where the road prism exhibits signs of distress, such as, for example:

(1) cracks or scarps within original ground above the top of cut slopes, in the road surface, or within the fill slope on the downslope side of the road; and

(2) significant zones of ground water seepage such that localized failure of cut or fill slopes are expected. Alternatively, installation of retaining wall structures may be needed to reduce excavation, contain bank material, or prevent slope failure. The significance of observed seepage zones might dictate application of other special measures to reduce the likelihood of slope failure during construction and over the operating life of the road.

c Consider the need to obtain advice from a Specialist for cut heights greater than 6 m, or where the ground slopes away from the edge of the excavation at a slope steeper than a ratio of 3 horizontal to 1 vertical. The advantages of steeper cuts may include: less area occupied by road; less excavated material; less side cast; and shorter slope lengths exposed to erosion processes. The disadvantages of steeper cuts may include: increased difficulty to establish vegetation; increased chance of slope ravelling, tension crack development and slope failure; and increased road maintenance costs. The disadvantages of steeper cut slopes can be reduced if high banks are avoided.

d Erosion control may be particularly problematic for slopes composed of sand, silty sands, or silts. Consider the need for erosion protection measures for cut slopes, fill slopes, and ditches, such as revegetation, soil bioengineering and biotechnical slope stabilization techniques, riprap, or other special slope treatments.

f If potential problems are anticipated for rock slopes either during design, construction, maintenance, or deactivation, consult with a geotechnical engineer or other rock slope specialist. It may be necessary to address the need for special rock slope stabilization measures (e.g., rockfall catch ditch, wire mesh slope protection, shotcrete, rock bolts).

(e) U.S. Department of Agriculture, Forest Service (USFS)

◆*LOW-VOLUME ROADS Engineering -Best Management Practices Field Guide*, The U.S. Department of Agriculture (USDA) Forest Service (Keller and Sherar, 2003)

These slope gradients correspond to the soil and rock classifications used by the U.S. Department of Agriculture (USDA) Forest Service (USFS) to specify cut slope gradients in forest road design (Table 4.2.18). This classification is based on experience and constructability.

Table 4.2.18 General Stability Slope Values by Soil Type (Keller and Sherar, 2003)

Table 11.1 COMMON STABLE SLOPE RATIOS FOR VARYING SOIL/ROCK CONDITIONS

Soil/Rock Condition	Slope Ratio (Hor:Vert)
Most rock	1/4:1 to 1/2:1
Very well cemented soils	1/4:1 to 1/2:1
Most in-place soils	3/41 to 1:1
Very fractured rock	1:1 to 1 ½:1
Loose coarse granular soils	1 ½:1
Heavy clay soils	2:1 to 3:1
Soft clay rich zones or wet seepage areas	2:1 to 3:1
Fills of most soils	1 1/2:1 to 2:1
Fills of hard, angular rock	1 1/3:1
Low cuts and fills (	2:1 or flatter (for revegetation)

(f) Bhutan

◆*Guideline on use of Standard Work Items for Common Road Works*

These are standards established by the Roads Division of Bhutan's Ministry of Works and Human Settlement (MoWHS) (*Guideline on the Use of Standard Work Items for Common Road Works* (MoWHS, 2010)). They constitute a classification system for specifying the slopes of cut slopes (Table 4.2.19).

Table 4. 2.19 Rock Classification in Bhutan (Fracture aspect indicated as V:H) (MoWHS, 2010)

	Primary National Highway Secondary National Highway Farm Road Dzongkhag Road
Ordinary Soil	1 : 1.0
Hard Soil	1 : 0.5
Soft Rock	1 : 0.25
Hard Rock	1 : 0.15

(g) Tajikistan

The standards specified by the Soviet Union's "Party Regional Committee for Construction" in 1986 (SNIP 2.05.02-85) were translated into Tajik and are still used as Tajikistan's standards (GNiP PT 32-02-2012, KMC, 2012) (KMC: Tajik abbreviation for the Committee for Construction and Architecture under the Government of the Republic of Tajikistan). This is a rock classification system for determining cut slope gradients and cut heights (Table 4.2.20). It is a classification based on experience and constructability. The abbreviation "GNiP" stands for the Russian acronym for "Urban Planning (Urban Development) Standards and Regulations."

Table 4. 2.20: Rock Classification in Tajikistan (Aspect ratio denoted as V:H) (KMC, 2012)

Soil	Height of slope, m	The largest Steepness of slopes
Rock:		
Slightly weathering	Up to 16	1 : 0.2
Easily airy:		
Non-softening	Up to 16	1 : 0.5 – 1 : 1.5
Softenable	Up to 6	1 : 1
	More than 6 to 12	1 : 1.5
Large-clastic	Up to 12	1 : 1 – 1 : 1.5
Sandy, clayey, homogeneous, solid, semi-solid and turgid-like consistency	Up to 12	1 : 1.5
Sands of small dune	More than 2	1 : 4
	From 2 to 12	1 : 2
Loess	Up to 12	1 : 0.1 – 1 : 0.5 1 : 0.5 – 1 : 1.5

Notes: 1. Above the slope is the steepness of slopes in the arid zone, below the line - outside the arid zone.
2. Vertical slopes are allowed in rocky weakly weathered soils.
3. In areas with vegetation-bound sands, the greatest steepness is allowed at a slope height of up to 12 m to take 1: 2.
4. The height of the slope of the recess is determined by the difference in the marks of the upper and lower slope of the slope. In the presence of slanting when using this table, a downward slope is taken into account.

(h) Tanzania

◆*Road Geometric Design Manual* (MoW,2011)

The standards developed by Tanzania's Ministry of Works (MoW) for road design (*Road Geometric Design Manual* (MoW, 2011)) specify the stable gradients for each rock type.

Table 4. 2.21 Rock Classification in Tanzania (Aspect Ratio: V:H) (MoW, 2011)

Type of material	Slope (Vertical : Horizontal)
Hard Rock	1:1 to 4:1
Decomposed Rock and/Compact lateritic soils	1:2 to 1:1
Ordinary soils	1:2 to 1:1.5

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4. 2. 2 Differences in Standard Cut Slope Grades for Specific Geology

(1) Issue

In Japan, separate standard cut slope gradients are established for specific geological conditions. These apply to special geological conditions where the general standard cut slope gradient cannot be applied for various reasons. While special geological conditions also exist in other countries, standard cut slope gradients for these specific conditions are often not established.

(2) Explanation

In Japan, materials such as weathered granite (masa-dō), pumice and scoria (shirasu), and serpentine are classified as special geological formations, for which separate standard cut slope gradients are established.

Unlike ordinary soil, weathered granite (masa-dō) has the characteristic of gradually changing weathering degree with depth, and the appropriate cut slope gradient varies depending on the weathering degree. While the upper limit for standard sandy soil slopes is 10m, strongly weathered masa-dō can accommodate slopes up to approximately 30m. Weathered granite approaching masa-dō characteristics may even permit vertical heights of around 50m. Furthermore, the standard cut slope gradient varies significantly with slope height.

Similarly, separate standard cut slope gradients are established for shirasu and serpentinite, primarily because the general standard cut slope gradient is often inapplicable in these cases. The reasons why masa-dō, shirasu, and serpentinite are classified as special soils in Japan are outlined below.

The reasons why strongly weathered granite, pumice and scoria, and serpentine are classified as special geologic conditions where the standard cut slope gradient cannot be applied are considered to be as follows.

Table 4.2.22 Special Geological Features in Japan and Their Characteristics

	Reasons for classification as special geological conditions where standard cut slope gradients cannot be applied
Weathered Granite (Strongly Weathered Granite)	・ Depending on the degree of weathering, it exhibits various properties ranging from weathered rock to sandy soil and cohesive soil. Since properties differ at each weathering stage, the standard cut slope gradient must be adjusted according to the degree of weathering. ・ Granite that has undergone strong weathering and become masa-dō is susceptible to erosion by surface water, making it prone to sediment loss.
Shirasu	・ Refers to the non- or weakly cemented portions of pyroclastic flow deposits originating from Quaternary volcanic activity throughout southern Kyushu, as well as their secondary deposits. ・ Its engineering properties vary depending on the degree of cementation and the depositional environment. ・ Engineering properties vary significantly based on “hardness,” necessitating adjustments to standard cut slope gradients according to the hardness of the underlying shirasu. ・ Particularly vulnerable to water erosion, requiring adjustments to standard cut slope gradients based on the presence of seepage water and necessitating drainage measures.
Serpentine rock	・ Due to stress release from cutting, followed by repeated drying and wetting cycles or freeze-thaw cycles, the surface layer of the slope gradually becomes prone to soil erosion. Therefore, it is necessary to ensure a stable slope gradient that does not collapse even as weathering progresses. ・ Fractures develop easily, and clay formation occurs along these fractures as weathering progresses.

To summarize the reasons outlined above, we consider that the following conditions should be classified as

special geological conditions, requiring separate standard cut slope gradients to be established.

- The weathered rock exhibits various conditions depending on the degree of weathering (weathered rock to sandy soil, cohesive soil).
- Properties such as susceptibility to weathering vary depending on various conditions.
- Weathering progresses rapidly due to environmental changes.
- Stability against water erosion is low.

(3) Actions to Take

Understand why specific geological conditions were selected in Japan and investigate whether similar problematic geological conditions exist in the country during cut slopes. If such geological conditions are present in the target area, the cut slope gradient must be determined based on a stability assessment that considers the properties of that specific geological formation.

(4) Reference Materials

Table 4.2.23 shows standard cut slope gradients for special geological conditions (geological conditions rarely seen in Japan) in various countries.

Table 4. 2.23 Standard Cut Slope Grades for Special Geology in Various Countries (Reference)

Soil Classification		Cut Height (m)	Slope (V:H)	Standard
English Name	Japanese Name			
Loess	Yellow Soil	<12	1:1.0 ~ 1:1.5	Tajikistan (Soviet Union) (KMC,2012)
		>12	1:1.5	
Laterite	Laterite	No specification	1:0.25 ～1:0.125*2	India (IRC, 1998)
Moorum	Moorum*1	No specification	1:1.0 ～1:0.5*3	
Marl	Marl*4	<10	1:1.0	Ethiopia (ERA,2013)
		10 – 15	1:1.25	
		15 - 20	1:1.5	
Red clay (Residual soil)	Brown clay (Residual soil)*5	Not specified	1:1.0	
Compact lateritic soils	Laterite*6	Not specified	1:1.0 ~ 1:2.0	Tanzania (MoW, 2011)

*1: Moorum/Murrum refers to tropical brown soils (red soil), a material commonly used in construction. It is distinct from laterite.
*2: Laterite is classified in the same category as Ordinary/Soft rock
*3: Murrum is classified in the same category as Ordinary/Heavy soil.
*4: The same criteria as for weakly weathered mudstone with numerous joints apply.
*5: The criteria shown here are based on actual construction cases and are intended as reference values.
*6: Dense laterite is treated as strongly weathered rock, not Ordinary soils.

<Sources>

Tajikistan: KMC (2012): *Automobile Roads/Highways* (GNiP PT 32-02-2012).
India: IRC (1998): *Hill Road Manual* (IRC SP 48-1998).
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Tanzania: MoW (2011): *Road Geometric Design Manual*.

4. 2. 3 Slope Protection Works and Drainage Measures After Cutting

(1) Issues

Generally, standard cut slope gradients are set assuming surface drainage measures and minimal erosion control measures are implemented on cut slopes. However, in many other countries, cut slopes composed of soil or highly weathered rock are often left untreated, frequently without any slope protection works or slope drainage works installed. Consequently, slope failures and instability are significant, making slope stabilization a major issues. Furthermore, a lack of understanding regarding the necessity of even minimal slope protection works and drainage measures is also a problem.

(2) Explanation

In countries or regions with extremely low rainfall, slope protection works or drainage works may not be necessary even for soil or highly weathered rock. However, these become necessary when a certain amount of rainfall occurs. Unprotected slopes, left in their natural state, become increasingly unstable over time due to surface erosion and weathering progression, making them prone to phenomena like collapse. In other words, neglecting minor slope issues such as erosion heightens the risk of escalation into larger slope disasters, leading to soaring countermeasure costs and prolonged road traffic restrictions. Therefore, to prevent such slope instability, surface water control measures and slope protection works must be implemented to safeguard the slopes.

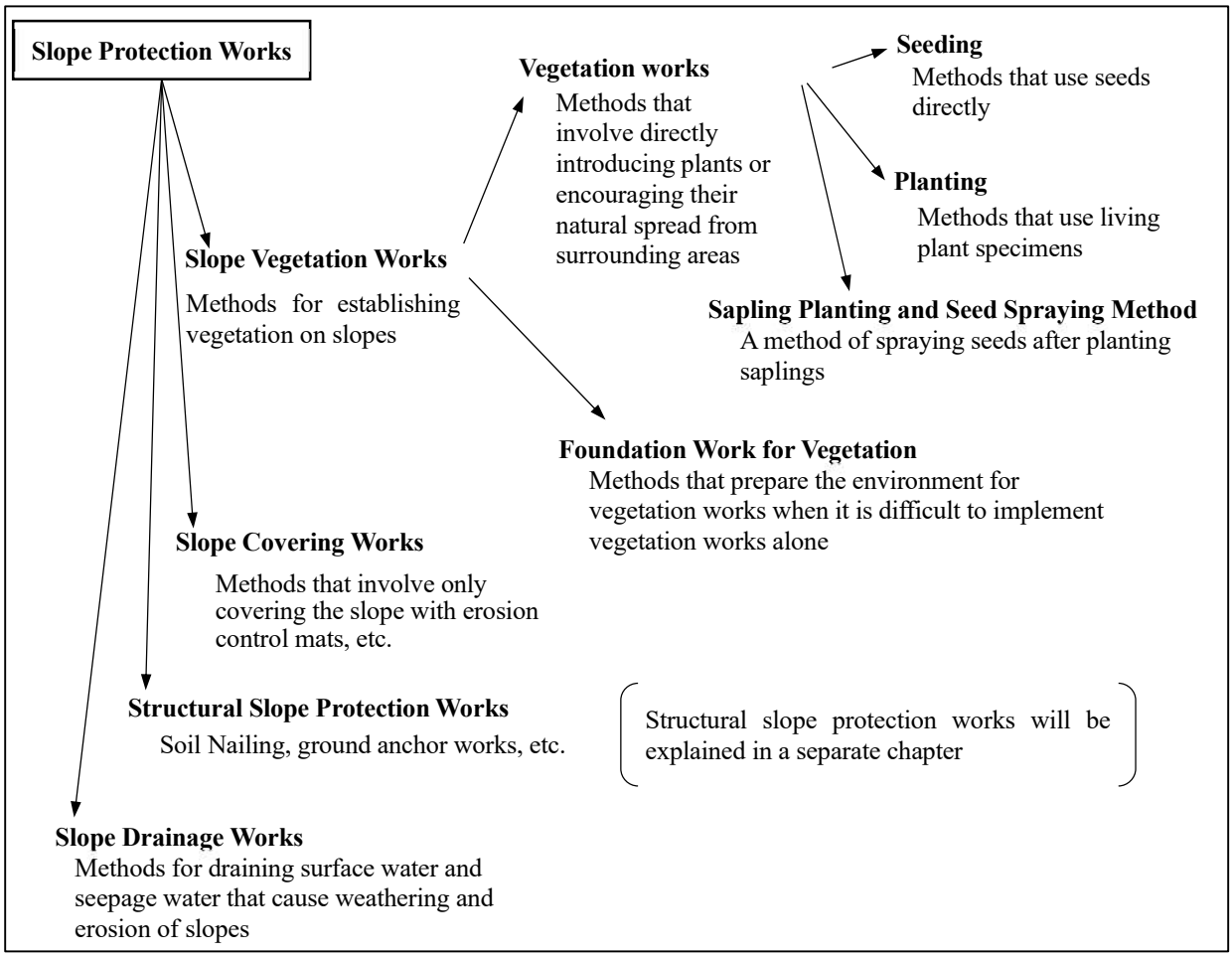


Figure 4.2.7 Types of Slope Protection Works and Surface Water Control Measures Required for Cut Earthworks

Figure 4.2.7 shows the types of slope protection works. Among these, slope greening, slope covering, and slope drainage are the minimum required slope protection works for cut slopes. The functions of these slope protection works are explained in Section 4.4.4 “Preventing Slope Erosion.” Structural slope protection works are explained in a separate chapter.

Slope greening and slope covering suppress surface erosion by covering the slope with vegetation or mats, thereby reducing surface runoff and binding soil particles. In cold regions, they also prevent collapse caused by freeze-thaw cycles. They can also prevent surface collapse caused by the intrusion of trees and other vegetation, offering various slope stabilization effects. Furthermore, drainage works are effective for rapidly discharging rainwater off the site, which is a cause of slope erosion and failure.

(3) Actions to Take

It is necessary to ensure understanding of the necessity and importance of slope protection works.
Since the materials available for slope protection works vary by target country, locally implementable methods must be considered. For example, approaches such as utilizing locally readily available materials like timber (e.g., contour wattling, fencing works, slope mulching) as suitable slope protection methods for the target country, or locally manufacturing U-shaped channel components for drainage ditchs, are required. In some countries, hand-built channels using rubble masonry are common. However, rubble masonry channels are prone to cracking and leakage, failing to maintain their functionality. Therefore, even a single channel component requires ingenuity and adaptation.

4. 3 Design Considerations

4. 3. 1 Utilizing Geological Distribution Maps and Topographic Maps

While geological maps may be available in some target countries, their accuracy can be poor, or they may not be publicly released at all. It is crucial to conduct extensive data collection, including literature and books, refer to existing geological distribution maps, and organize the topography and geology requiring prior attention.
In countries lacking topographic maps at practical scales, it is vital to create topographic maps using satellite imagery, UAVs, or detailed surveying, and to conduct terrain interpretation beforehand.

4. 3. 2 Importance of Preliminary Field Surveys

The geology and geological structures of the target site are extremely important for planning and designing cut-and-fill works. Therefore, it is necessary to directly verify the actual site conditions, using the information in “4.3.1 Utilization of Geological Distribution Maps and Topographic Maps,” “4.3.4 Geological Structures Requiring Caution,” and “4.3.5 Excavation Conditions Requiring Caution” as references, to understand aspects of the site that cannot be determined solely through analysis of existing data.

4. 3. 3 Terrain and Geology Requiring Caution

Natural slopes are extremely complex and heterogeneous in composition, containing heterogeneous soil and rock masses, geological discontinuities such as joints and faults, and weathered or altered sections. Cut slopes rarely feature homogeneous, solid geology throughout the entire slope. They often incorporate weak zones like faults, frequently harboring potential stability issues. Therefore, detailed geological conditions, groundwater conditions, and potential problems at the planned cut slope location must be identified through preliminary geological surveys and incorporated into the design.

The following terrain and geological conditions require special attention when designing cut slopes. These are explained in Section 4.4.6, “Cut Slopes Requiring Particular Attention.”

1) Cutting works in dormant landslide
2) Cutting works on talus, colluvial deposits, and highly weathered slopes
3) Cutting works in sandy soils and other soil types particularly susceptible to erosion
4) Cutting works in expansive rock or rapidly weathering geological formations

4. 3. 4 Geological Structures Requiring Caution

The geological structures requiring caution when designing cut slopes are listed below. These are explained in the section “4.4.6 Cut Slopes Requiring Particular Caution.”

1) Slope gradient for rock with numerous fractures
2) In case fractures form a dip slope

4. 3. 5 Excavation Conditions Requiring Caution

(1) Cutting at Locations with High Groundwater Levels or Abundant Seepage

The majority of slope failures are directly or indirectly influenced by groundwater. Therefore, when cutting at locations with high groundwater levels or abundant seepage, depending on geological conditions, the slope may possess unstable elements. In such cases, either the slope gradient should be made gentler than the gradient

determined by standard stability analysis, or drainage measures should be implemented to create a slope condition less prone to failure.

Design underground drainage works (groundwater drainage ditches, gabion structures, horizontal drainage holes) as drainage measures to collect seepage water from the slope and groundwater near the ground surface and drain it away from the slope.

(2) Long Slopes

Long slopes pose a significant risk of major disasters if they collapse. Furthermore, modifying the cut (re-cutting) after excavation has progressed is disadvantageous both economically and in terms of constructability. Therefore, it is desirable to reduce the amount of cut modification beforehand through minor route adjustments. If a long slope is unavoidable, it is advisable to design it with greater safety margins compared to a standard cut. For example, with expansive rock or rapidly weathering rock, adopt a gentler slope gradient than that determined by standard stability analysis. This accounts for secondary strength reduction due to stress release after cutting and rapid weakening from accelerated weathering, ensuring slope safety even if weakening progresses rapidly. Additionally, for slope terraces, wide terraces approximately 3m to 4m in width should be provided every 20m to 30m vertical height.

References

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

4. 4 Example Descriptions of Items Requiring Attention When Creating Technical Standards, etc.

4. 4. 1 Standard Gradient for Cut Slopes

Standard gradient values for cut slopes are established for use during preliminary design in the preliminary investigation phase.

While design for cut slope stability should ideally be based on detailed investigation results to determine stability, if these standard values are used in detailed design, geological conditions and groundwater conditions must be fully considered.

【Explanation】

When designing cut slopes, the slope gradient is determined based on investigations including soil testing, surrounding topography and geological conditions, past disaster history, and the actual state of similar slopes. Fundamentally, all cut slope gradients require individual stability assessments based on detailed investigation results.

Where preliminary investigations indicate favorable geological conditions, standard cut slope gradients may be referenced to determine the slope gradient. Appropriate cut slope gradients must be determined considering not only local geological and groundwater conditions but also meteorological factors and the magnitude of potential triggers such as earthquakes. Standard cut slope gradients for typical geological conditions are shown in Table 4.4.1.

For typical geological conditions, the cut slope gradient has a range of standard values even within a single rock mass classification, such as hard rock or soft rock. The specific value within this range to be adopted is determined based on the results of topographical and geological surveys. Additionally, consideration of the cut slope gradient must account for any restrictions imposed by the boundaries of the acquired land.

Table 4.4.1 shows the standard cut slope gradients for Japan, which are based on the premise of the minimum necessary slope protection. When slope protection structures are used in combination, gradients different from those in Table 4.4.1 may be adopted.

Furthermore, during cut construction, attention must be paid to geological changes. If fault zones, dikes, or discontinuities (joints, bedding planes, foliation, fault planes) that dip daylight into the slope appear, other than the initially anticipated geology, a comparison with the original design must be performed. Design changes, such as to the slope gradient, must then be made.

<Supplementary Explanation for Table 4.4.1: The following applies when adopting Japan's standard slope gradient table>

For determining standard cut slope gradients, the ground classification is as follows: material excavatable directly by bulldozer is classified as earth and sand; material primarily requiring ripper work is classified as soft rock; and material requiring blasting for excavation is classified as hard rock. The boundaries between these categories show some correlation with elastic wave velocity when using the following criteria as a guide: the soft rock-hard rock boundary is based on the feasibility of ripper work with a 32-ton class bulldozer, and the soft rock-soil boundary is based on the feasibility of excavation using a dozer blade with a 21-ton class bulldozer (Table 4.4.2).

Sandy soils are classified as either “dense” or “non-dense,” with sandy soils having an N-value greater than 20 to 30 being judged as “dense.”

Table 4.4.1 Classification of Natural Ground and Standard Slope Grades in Japanese Slope Stabilization Measures (Example with Subdivided Rock Categories)
(Based on *Design Guidelines: Highways* (MLIT, 2020), with some additions)

Ground Classification		Cut Height	Slope (V:H) (JRA, 2009b)	Standard Slope Value (V:H)
Hard Rock※			1:0.3 -1:0.8	Hard Rock 1:0.3 Medium Hard Rock 1:0.5
Soft Rock※			1:0.5 - 1:1.2	Soft Rock 1:0.5 - 1:0.7 Weathered Rock 1:0.7 - 1:1.2
Sand	Non-compact, poorly distributed grain size		1:1.5 -	1:1.8
Sandy Soil	Compacted material	5m or less	1:0.8 - 1:1.0	1:1.0
		5 - 10m	1:1.0 - 1:1.2	
	Loose material	5m or less	1:1.0 - 1:1.2	1:1.2
		5 - 10m	1:1.2 - 1:1.5	
Sandy soil mixed with gravel or boulders	Compact, or with good particle size distribution	10m or less	1:0.8 - 1:1.0	1:1.0
		10 - 15m	1:1.0 - 1:1.2	
	Loose material or material with poor grain size distribution	10m or less	1:1.0 - 1:1.2	1:1.2
		10 - 15m	1:1.2 - 1:1.5	
Cohesive soil		10m or less	1:0.8 - 1:1.2	1:1.2
Clayey soil containing boulders or cobbles		5m or less	1:1.0 - 1:1.2	1:1.2
		5 - 10m	1:1.2 - 1:1.5	

(Note) ※**: The classification of hard rock and soft rock is based on the difficulty of earthwork, not rock type, according to a classification unique to Japan.**
(Table 4.4.1 shows values for Japan. While using these standard values as a reference, it is necessary to establish original standard cut slope gradients for each country based on their own standard values and construction experience in that country.)

Table 4.4.2 Guidelines for Classifying Ground Based on Workability with Heavy Machinery and In-Situ Elastic Wave Velocity
(Compiled from JRA (1986) and JRA (2009))

Ground Classification	Criteria based on construction feasibility with heavy machinery, JRA (1986)	Ground elastic wave velocity, JRA (2009a)
Soil	Excavation possible using a 21-ton class bulldozer's dozer blade	Approximately 700 m/s
Soft Rock	Excavation using a 21-ton class bulldozer's dozer blade is difficult Ripper work possible using a 32-ton class bulldozer	Approximately 2.0 km/s or less
Hard Rock	Ripper work using a 32-ton class bulldozer is difficult	Approximately 2.0 km/s or more

References

MLIT (2020): *Design Guidelines: Highways*, Ministry of Land, Infrastructure, Transport and Tourism(MLIT), Hokuriku Regional Development Bureau.

JRA (1986): *Guidelines for Highway Earthwork: Soil Investigation*, Japan Road Association (JRA)

JRA (2009): *Outline for Highway Earthwork*, Japan Road Association (JRA), 416p.

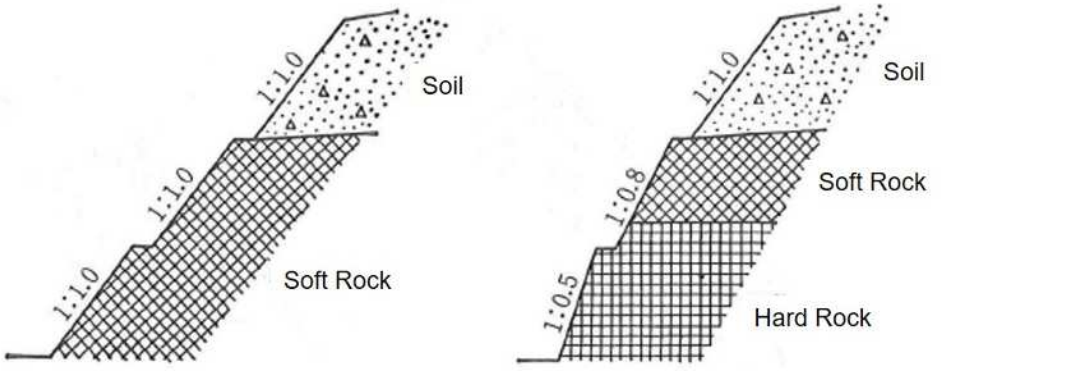
4. 4. 2 Slope Geometry

Cut slopes generally take the following forms, selected based on the distribution of geological conditions.
1) Single-gradient slope
2) Slope with gradient varying according to soil and rock properties

【Explanation】

When geological conditions are generally uniform both vertically and horizontally, a single slope gradient is typically adopted. Even if geological conditions vary, a single slope gradient is acceptable if it is set according to the geological conditions requiring the gentlest slope.

In normal cases, the ground becomes harder with depth below the surface, allowing steeper slopes to remain stable. Therefore, slopes with gradients varying according to soil and rock conditions are adopted. In such cases, where the standard slope height for the target geology is exceeded or where significant geological changes occur, terraces are provided to vary the slope gradient above and below these points. Near the toe of cut slopes, vegetation is difficult to establish, and generally, loose soil and weathered rock are present, making the area susceptible to erosion and collapse. Therefore, rounding is performed to prevent toe failure as much as possible and to improve the landscape.



(a) Example of a slope with a uniform gradient

(b) Example of a slope with a gradient varied according to soil and rock types

Figure 4.4.1 Schematic Diagram of Ground Conditions and Slope Geometry (JRA, 2009)(Slope value V:H)

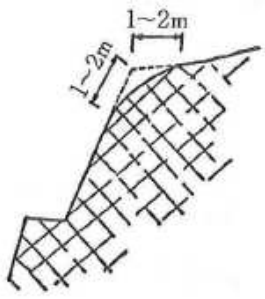


Figure 4.4.2 Conceptual Diagram of Shoulder Rounding (JRA, 2009)

References

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

4. 4. 3 Terraces

Terraces are installed to limit the height of a single slope and prevent slope failure, while also facilitating surface water drainage and inspections during maintenance. When installing them, consider the following points:
1) Terrace location and width
2) Terrace cross-slope gradient

【Explanation】

On slopes with high cut heights, a small terrace at least 2 meters wide should be installed mid-slope. This prevents slope failure and maintains slope stability by limiting the height of a single cut. If the terrace width is too narrow, the slope segmentation effect weakens, increasing the risk of large-scale failure extending beyond the terrace.

Furthermore, if a continuous slope is constructed without intermediate steps, the flow rate and velocity of surface water at the lower slope section increase compared to a stepped slope. This heightens scouring forces, destabilizing the slope. Therefore, installing nearly horizontal intermediate steps mid-slope reduces flow velocity, thereby maintaining slope stability.

(1) Position and Width of Terraces

Determine the position of terraces so that the height of a single step does not exceed geological limits.
On slopes composed of the same geological material, terraces may be mechanically spaced at equal intervals. However, where geological conditions differ, it is desirable to position them at boundaries such as those between soil and rock, or permeable and impermeable layers, considering seepage (Figure 4.4.3).

The width of the terraces should generally be at least 2 m, considering their use as inspection walkways and scaffolding for repairs. However, for long slopes, maintenance is more critical than for standard slopes. Therefore, it is desirable to install wider terraces, approximately 3 m to 4 m wide, every 20 m to 30 m of vertical height.

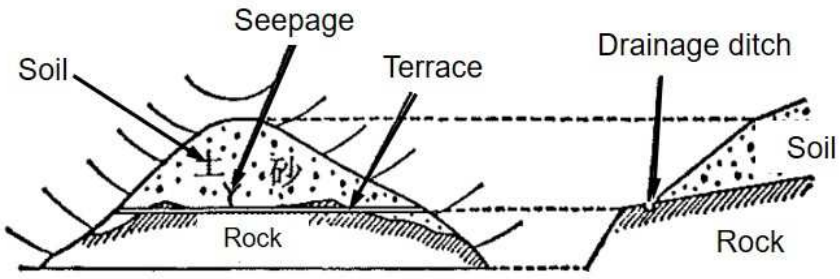


Figure 4.4.3 Position of Terraces (JRA, 2009)

(2) Cross-slope of Terraces

For small-scale slopes with minimal maintenance issues where drainage facilities are not installed on terraces, the cross-slope of the terrace should be designed to allow water to flow toward the toe (approximately 5–10%).
Where slope detachment is anticipated to be significant or where terrace shoulders are susceptible to erosion, drainage facilities should be installed on the terrace. To ensure water collects in the drainage ditch, the terrace should be constructed with a reverse slope.

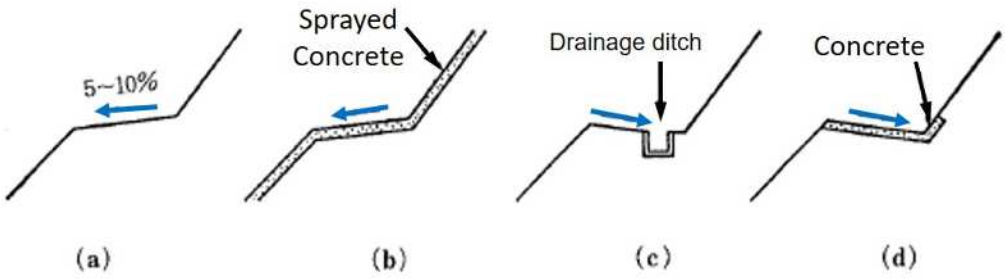


Figure 4.4.4 Cross-slope of a small terrace (JRA, 2009)

References

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

4. 4. 4 Slope Erosion Prevention

To suppress the destabilization of cut slopes due to erosion, etc., protect the slopes with slope protection works and install surface water drainage facilities.

【Explanation】

Cut slopes composed of soil or highly weathered rock are susceptible to erosion and other impacts. To protect slopes from rainwater erosion and prevent minor slope disturbances such as surface deformation, slope protection works like slope greening or slope covering are implemented.

Furthermore, the primary factor leading to slope failure is the action of water. Therefore, measures against water are critically important to ensure slope stability. This requires not only permanent drainage works but also sufficient consideration for drainage measures during construction.

(1) Methods of Slope Protection

Cut slopes become increasingly unstable as surface water runoff causes erosion of the topsoil, potentially leading to major failures such as large-scale collapses. To prevent such slope instability, surface water control measures and slope protection works are employed to protect the slope. The types of slope protection works are shown in Figure 4.4.5.

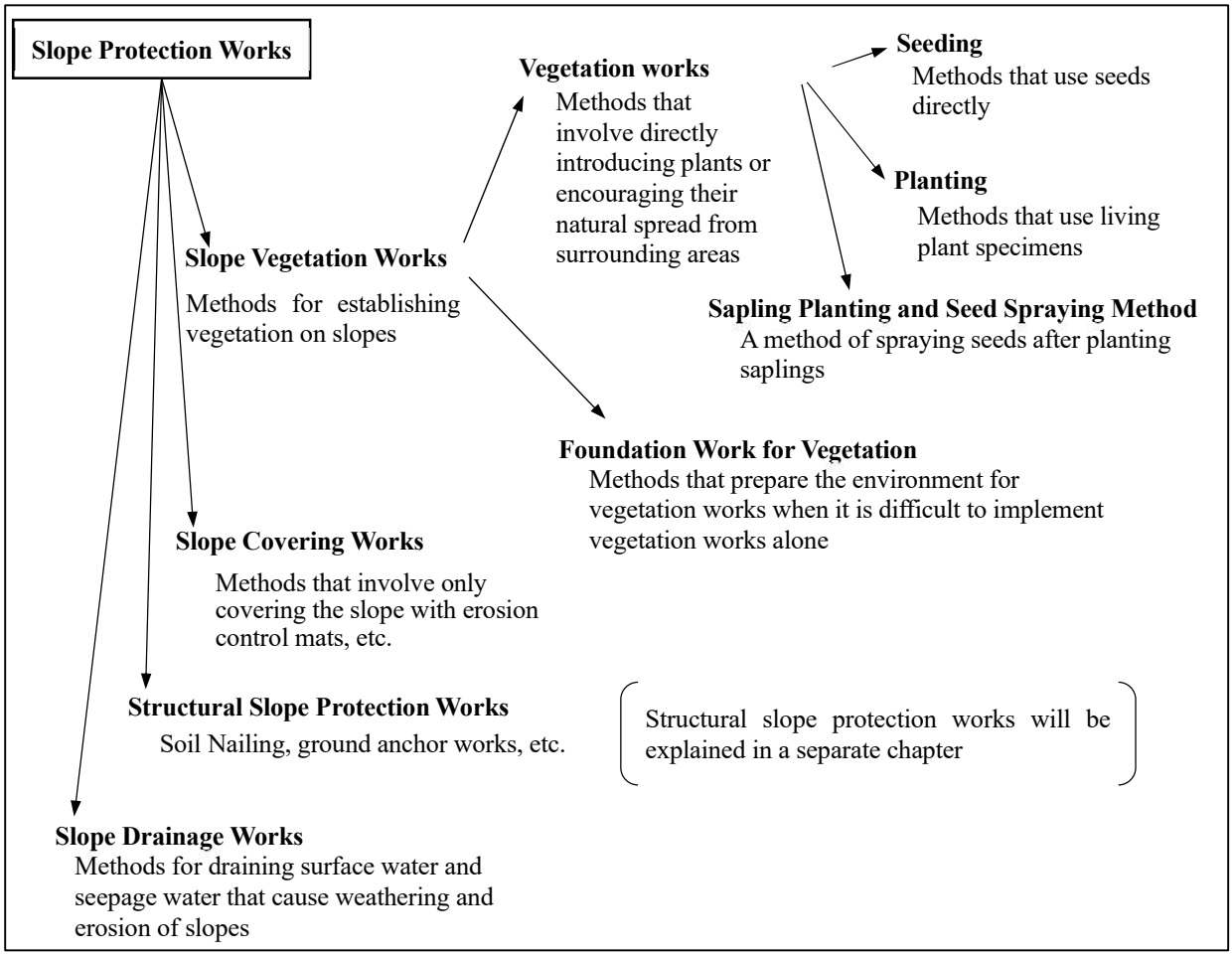


Figure 4.4.5 Types of Slope Protection Works and Surface Water Control Measures

Slope protection works essential for cut slopes include slope greening, slope covering, and slope drainage. Slope greening is a general term for methods that protect slopes by promoting plant growth on them. Covering slopes with vegetation prevents erosion caused by rainwater. Furthermore, establishing vegetation on slopes that harmonizes with the surrounding natural environment helps preserve the natural environment and landscape. Slope greening consists of vegetation works, which introduces plants to the slope, and revegetation foundation works, which prepares the plant growth environment using structures or other means. When vegetation works alone cannot be implemented, such as on steep slopes, vegetation works and revegetation foundation works are combined.

Slope covering works involve covering slopes solely with erosion control mats that do not contain plant seeds. By tightly securing well-aerated and permeable mats to the slope, these works prevent soil runoff caused by rainwater and suppress collapse due to freezing and frost heave in cold and high-altitude regions, thereby stabilizing the slope.

Additionally, as a slope protection method suitable for the target country, there are mountain slope works (contour wattling, fence works: Table 4.4.6; slope mulching: Table 4.4.7, etc.) that utilize locally available materials such as timber.

＜Column 1＞

The slope protection techniques and drainage ditch materials that can be introduced vary by target country, so locally feasible methods must be considered. For example, alternatives to erosion control mats may need to be sought, or locally manufactured U-shaped channel components may be required. Furthermore, while hand-built stone-masonry channels are common in some countries, these structures easily crack and leak, failing to maintain their functionality. This underscores the need for ingenuity even in selecting a single channel component.

＜Column 2＞

Since suitable plants vary depending on the target country's climate and other conditions, it is necessary to research plants appropriate for vegetation works and similar applications beforehand.

(2) Surface Water Treatment Methods

Since many slope failures are caused by erosion from surface water or seepage, water management is crucial for ensuring slope stability. Therefore, shoulder drainage ditches, terrace drainage ditches, and vertical drainage ditches are installed to promptly drain surface runoff from rainfall away from the slope.

Additionally, where seepage appears, subsurface drainage ditches, gabion structures, and horizontal drainage holes are installed as necessary.

Table 4.4.3 Types of Slope Drainage Structures

Type	Description
Surface drainage works	**Focusing on surface water**
Slope shoulder drainage ditch	Installed along the slope shoulder to prevent surface water from adjacent areas flowing onto the slope
Terrace drainage ditch (Fig. 4.4.6)	Installed at the bench level of the slope to drain rainwater falling on the slope
Longitudinal drainage ditch	Installed along the slope to direct water from the shoulder drainage ditch and bench drainage ditch to the channel at the toe of the slope
Subsurface Drainage Works	**Applies to seepage water and groundwater**
Subsurface Drainage ditch (Fig. 4.4.7)	Collect and drain groundwater, seepage, and seepage water near the surface. Determine the location and structure based on the groundwater level and seepage conditions. For manual excavation, use an inverted trapezoidal cross-section with a bottom width of 30 cm. Install gabions or similar structures within the excavated trench. Additionally, a drainage layer made of brushwood or crushed stone may be placed on the top or sides to prevent clogging, and a vinyl sheet may be laid at the bottom to prevent leakage.
Horizontal Drainage Holes (Fig. 4.4.8)	Installed to remove seepage water from slopes. The length is generally 2 meters or more. Holes are drilled using methods such as boring, and drainage pipes with strainer fittings are inserted. Since the hole openings are prone to scouring due to drainage, protection using a basket or similar device is desirable.

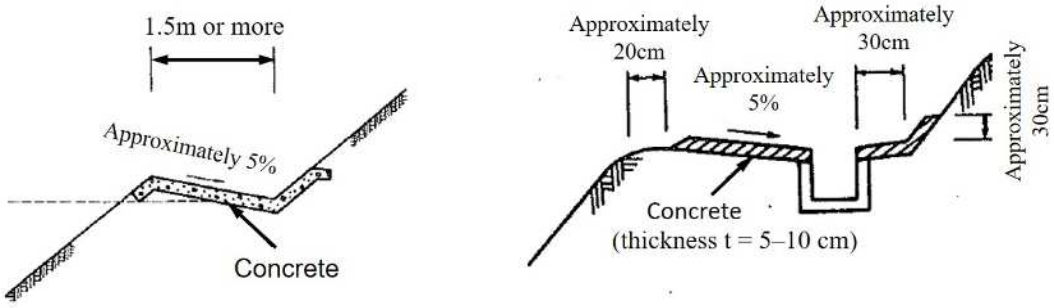


Figure 4.4.6 Examples of Small-Scale Drainage ditchs (Concrete Drainage ditch and Reinforced Concrete U-Channel) (JRA, 2009)

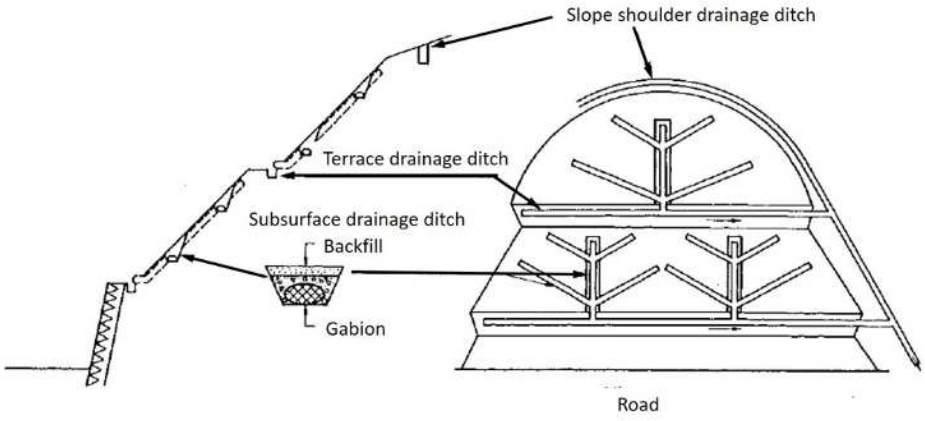


Figure 4.4.7 Example of Subsurface Drainage ditch (JRA, 2009)

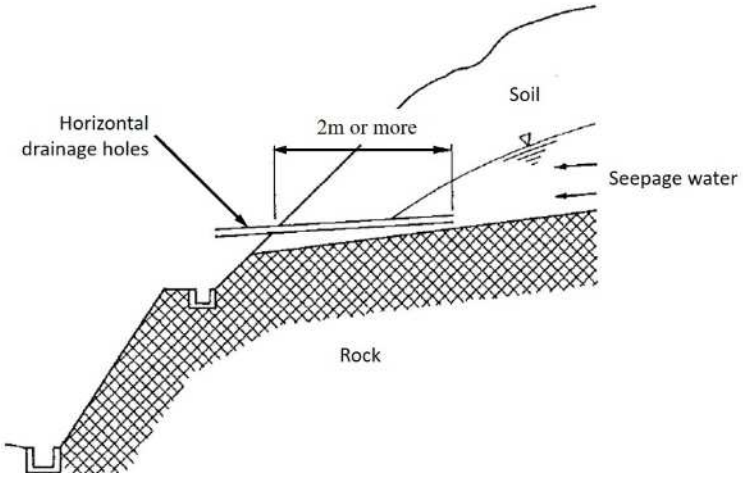


Figure 4.4.8 Example of Horizontal Drainage Holes (JRA, 2009)

【Reference Notes】

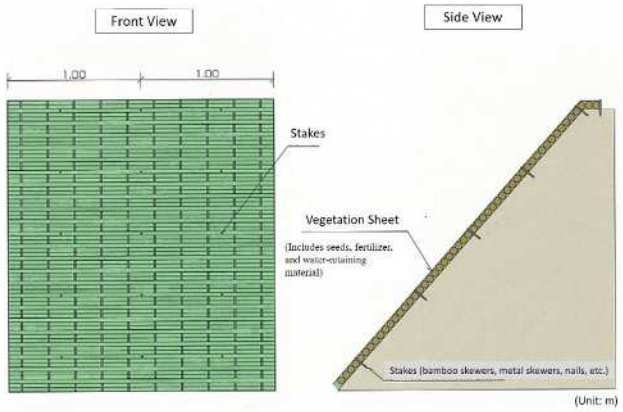
・Since maintaining the channel's functionality may sometimes prove difficult, it is advisable to consider installing rubber mats on the base of open/closed channels to ensure proper water flow, as well as locally manufacturing U-shaped channel components.

・Examples of simple slope protection are explained on the following pages.

Table 4.4.4 Vegetation works Case Study (1)

Seeding Work





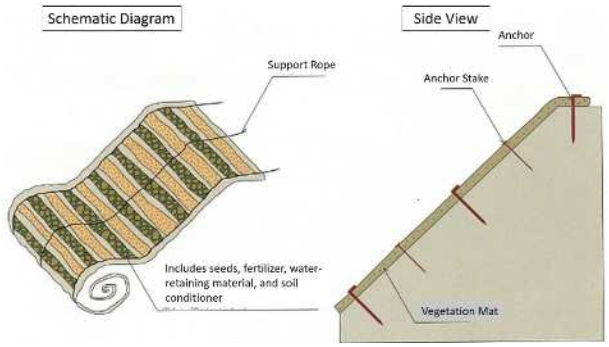
Vegetation Sheet works:

Attach covering material loaded with seeds and chemical fertilizer to the entire slope surface using stakes to ensure close contact. Covering materials include straw, matting, and non-woven fabric.

■Main Materials — Per 100 m^2

Name	Dimensions	Unit	Quantity	Description
Vegetation sheet	10m long × 1m wide	m^2	120	Includes seeds and high-quality chemical fertilizer
Stakes	150mm long	Pieces	500	Metal stakes





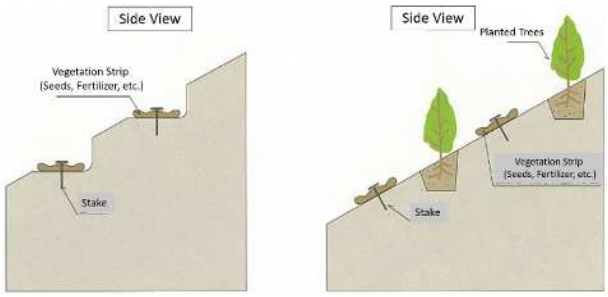
Vegetation mat works:

Compared to vegetation sheets, the thicker material provides superior coverage and excels at preventing surface erosion caused by rain immediately after installation.

■Main Materials — Per 100 m^2

Name	Dimensions	Unit	Quantity	Description
Vegetation Mat	10m long × 1m wide	m^2	120	Includes seeds, fertilizer, water-retaining material, and soil conditioner
Anchor	9 mm diameter, 200 mm long	Piece	184	
Fastening nails	with washers, 150 mm	Piece	422	





Vegetated Strip Works (Vegetated Strip Engineering):

Embedding strip-shaped cover materials loaded with seeds, fertilizer, etc., to achieve greening.

■Main Materials — Per 10 m

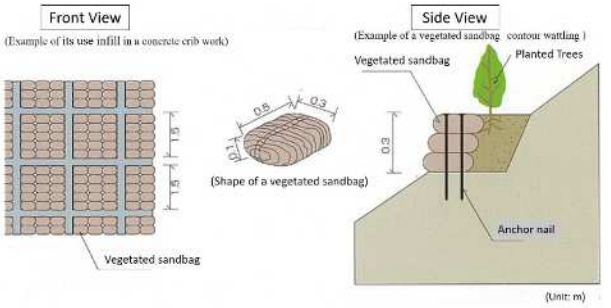
Name	Dimensions	Unit	Quantity	Description
Vegetation strip	50 cm long × 10 cm wide	sheet	20	Includes seeds and fertilizer
Stake	150 mm long	piece	40	

(Source: Supervised by the Forestry Agency: *Guide to Revegetation Work for Creating Nature* (JFECF, 1997))

Table 4.4.5 Vegetation works Case Study (2)

Seeding Work





Vegetated sandbag work:

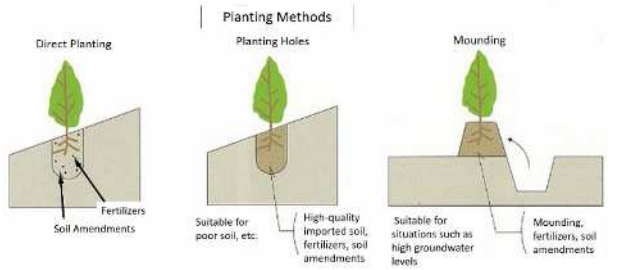
Green areas created by arranging soil bags filled with seeds and fertilizer in a surface or strip pattern.

■Main Materials — Per 10 m

Name	Dimensions	Unit	Quantity	Description
Vegetated sandbag	50 × 30 × 10cm	bag	60	Includes seeds, fertilizer, and water-retaining material
Sandbag	Humus	Pieces	0.5	For filling with soil
Anchor nail	150mm long	Pieces	40	

Planting works





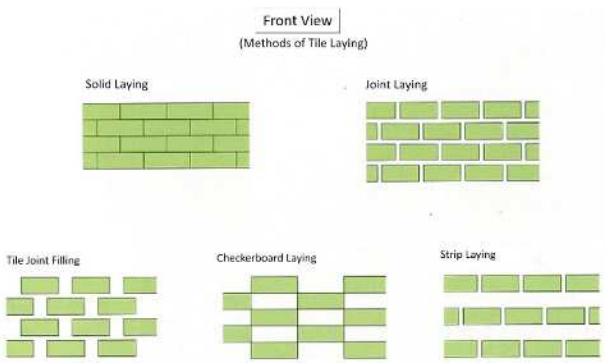
Planting works:

Plant seedlings at appropriate intervals and in an orderly arrangement to create greenery.

■Main Materials — Per 1 piece

Name	Dimensions	Unit	Quantity	Description
Saplings		Pieces	1	
Fertilizer	chemical fertilizer			Determined based on the tree species
	organic fertilizer	g	300–500	compost
Topsoil	humus-rich soil	m^3		Apply as needed





Sodding:

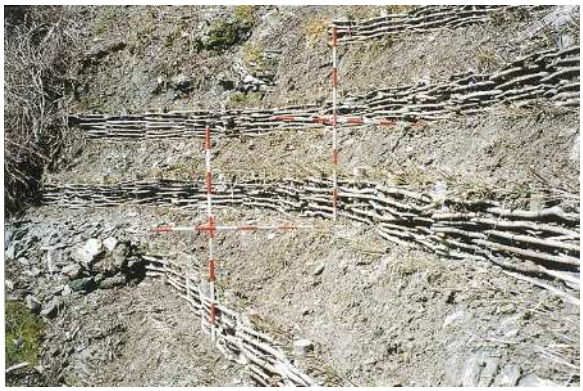
Attach turf to the ground surface to prevent soil erosion caused by surface water, etc.

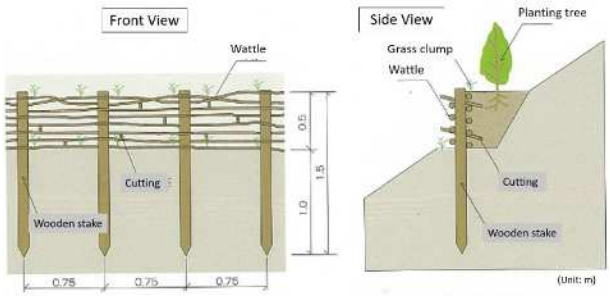
■Main Materials — Per 10 m^2

Name	Dimensions	Unit	Quantity	Description
Turf	36 × 14cm	sheet	198	Solid laying
Stake	Bamboo stake	Pieces	396	2 stakes per sheet
Topsoil	High-quality soil	m^3	0.2	Compost

(Source: Supervised by the Forestry Agency: *Guide to Revegetation Work for Creating Nature* (JFECF, 1997))

Table 4.4.6: Examples of Vegetation Foundation Works (1)





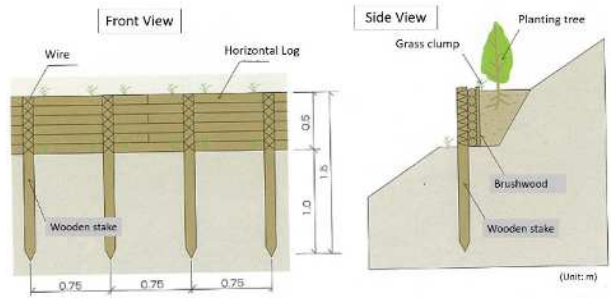
Wattle fence:

Drives stakes into the slope and weaves twigs to create the fence.

Main Materials — Per 10 m

Name	Dimensions	Unit	Quantity	Description
Wooden stake	Length: 1.5m, tip diameter: 9cm	Pieces	13	Spacing: 0.75 m
Wattle	Bamboo stake	bundle	5	Height of raised top of the fence: 0.5 m
Cutting	Length: 0.4m, diameter: 0.7~3cm	Pieces	50	Willow, deutzia, etc.
Grass clump	Stem length 30 cm, spacing 1 m, secured with rope	bundle	4	Used on both the top and base of the fence





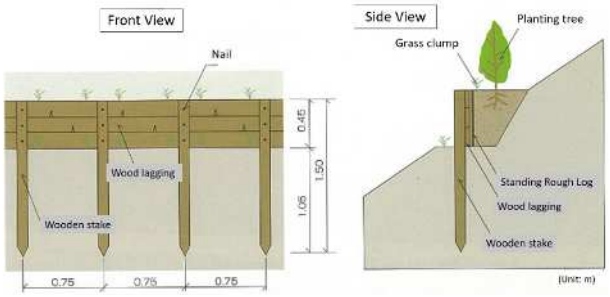
Log Fence:

Logs are used as wall material. They offer slightly greater strength and durability than the wattle fence method. The wall material is fixed to the uphill side of the posts to prevent topsoil movement.

Main Materials — Per 10 m

Name	Dimensions	Unit	Quantity	Description
Wooden stake	Length: 1.5m, tip diameter: 9cm	Pieces	13	Spacing: 0.75 m
Horizontal Log		Pieces	30	Height of raised top of the fence: 0.5 m
Grass clump	Stem length 30 cm, spacing 1 m, secured with rope	bundle	1.5	Used on both the top and base of the fence
Wire	Annealed Wire #12–#18	kg	3	





Plank fence:

Boards are used as wall material. While boards are less strong and durable than logs, they are lighter and easier to install.

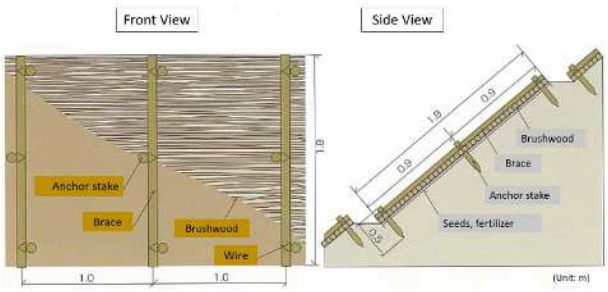
Main Materials — Per 10 m

Name	Dimensions	Unit	Quantity	Description
Wooden stake	Length: 1.5m, tip diameter: 9cm	Pieces	13	Spacing: 0.75 m
Wood lagging	Length 2.1m, width 15cm, thickness 1.5cm	Pieces	15	Height of raised top of the fence: 0.5 m
Grass clump	Stem length 30 cm, spacing 1 m, secured with rope	bundle	1.5	Used on both the top and base of the fence
Nail	Length 4.5 cm	kg	0.15	

(Source: Supervised by the Forestry Agency: *Guide to Revegetation Work for Creating Nature* (JFECF, 1997))

Table 4.4.7: Examples of Vegetation Foundation Works (2)





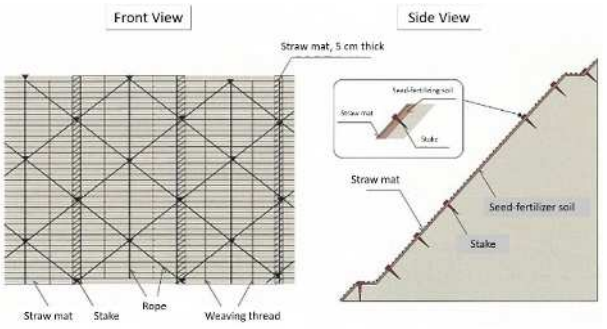
Brush mattress:

Lay brushwood on the slope surface and secure it with stakes. Covering the ground surface prevents topsoil erosion and the loss of seeds and other materials.

Main Materials — Per 180 m²

Name	Dimensions	Unit	Quantity	Description
Brushwood	Length 2 m or more, tied with 1 m of rope	bundle	4.5	
Brace	Length 1.8 m, diameter at the small end 5 cm or more	Pieces	10	
Anchor stake	Length 0.5 m, diameter at the small end 5 cm or more	bundle	30	
Wire	Annealed Wire #18	kg	0.17	





Straw matting:

Straw mats are used as the covering material. When straw is used as the covering material, it is called straw mulching. The materials are lightweight, making transportation easy and construction straightforward.

Main Materials — Per 10 m

Name	Dimensions	Unit	Quantity	Description
Wooden stake	Length 1.5 m, tip diameter 9 cm	Pieces	13	Spacing 0.75 m
Horizontal log	Length 3.6 m, tip diameter 3–8 cm	Pieces	30	Height of raised top of the fence: 0.5 m
Grass clumps	Root length 30 cm, spacing 1 m, secured with rope	bundle	1.5	Used on both the top and foundation surfaces
Wire	Annealed Wire #12–#18	kg	3	

(Source: Supervised by the Forestry Agency: *Guide to Revegetation Work for Creating Nature* (JFECF, 1997))

References

JFECF (1997): *Guide to Revegetation Work for Creating Nature: Regeneration and Creation of Greenery (Supervised by Forestry Agency)*, Japan Forest Engineering Consultants Foundation (JFECF), 244p.

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

4. 4. 5 Stability Evaluation of Cut Slopes

Design of cut slopes should ideally be based on stability evaluations conducted using detailed survey results. However, if no particular issues are identified during the preliminary survey stage, design may proceed using standard slope gradients and reference to existing cut slopes in the vicinity.

【Explanation】

Natural ground consists of heterogeneous soil and rock masses, geological discontinuities such as joints and faults, and weathered or altered zones, resulting in a complex and heterogeneous composition. Furthermore, due to rainfall, earthquakes, or long-term weathering, cut slopes gradually become unstable after construction.

Multiple disaster scenarios following cut slopes must be considered, and stability assessments must be conducted for each scenario. The method of stability assessment varies depending on the type of disaster being evaluated (rockfall, rock mass collapse, topping, circular slide, planar slide, etc.).

＜Column 3＞

Japan has established standard cut slope gradients developed through extensive experience, and this empirical technology has been prioritized in cut slope design. These gradients were created based on proven results showing that, within specified cut heights and under Japan's natural environment, major issues affecting protected structures like roads can be avoided. Since other countries lack such empirical results, they return to fundamentals, prioritizing individual stability assessments for each cut slope.

4. 4. 6 Cut slopes requiring special attention

Sections meeting the following conditions must be examined with particular care, and the slope gradient must be determined accordingly.
1) Dormant Landslide
2) Talus, colluvial deposits, or highly weathered slopes
3) Sandy soils or other soil types particularly susceptible to erosion
4) Expansive rocks or rapidly weathering rocks
5) Rocks with numerous fractures
6) Cases where fractures are dip slopes
7) Areas with high groundwater levels
8) Cases involving long embankment slopes

【Explanation】

(1) In the case of dormant landslide

Cutting into the toe or flank of dormant landslide poses a risk of triggering slow-moving landslides. Therefore, for active slow-moving landslide areas, the plan itself must be reconsidered. Alternatively, prior to construction, the distribution of dormant landslide in the target area must be investigated using methods such as aerial photo interpretation. This allows for the design of cuts that are less susceptible to slow-moving landslide activity and do not adversely affect slow-moving landslides (avoiding cuts at the slow-moving landslide terminus or flanks).

(2) For Talus, Collapsed Deposits, and Severely Weathered Slopes

Sediment in areas such as talus, debris flow deposits, mudflow deposits, slope failure scar terrain, and severely weathered slopes can become extremely unstable even in their natural state. This instability arises from reduced strength due to water saturation during rainfall, the generation of excess pore water pressure, or events like earthquakes, and may even lead to collapse. Countermeasures are necessary for such locations.

1) In case only surface layer collapse is anticipated

Prevent slope erosion through slope protection works and promptly drain surface runoff from precipitation via slope drainage works. Additionally, create wide steps below the colluvial deposits slope section to form a shape where collapsed debris accumulates easily in these steps if the colluvial deposits portion fails. The slope gradient of cut slopes in colluvial deposits or highly weathered layers should be set to the lower limit of the standard cut slope gradient range or even gentler.

2) In case large-scale failure is anticipated, involving not only the surface layer but also deep ground layers

Either perform large-scale earth removal, install sufficient groundwater drainage works, or reconsider the planned location.

(3) Sandy soils, etc., particularly vulnerable to erosion

Volcanic ash soils, weathered granite, mountain sand, and terrace gravel layers are susceptible to erosion by surface runoff, often leading to sediment loss. Therefore, shoulder drainage ditches and terrace drainage ditches should be installed to prevent surface water from flowing down the slope and to prevent slope erosion.

(4) In the case of expansive rocks or rapidly weathering rocks

1) In the case of expansive rocks

Expansive rocks containing large amounts of expansive clay minerals such as montmorillonite often weather rapidly due to stress release from cutting and subsequent repeated drying and wetting cycles. This leads to a significant decrease in consolidation after cutting, frequently causing collapse. In such cases, even slopes that appear hard and stable immediately after cutting may experience strength reduction in the future. Therefore, to account for potential slope weakening, gentler slopes than those determined by standard stability analysis should be adopted to ensure long-term slope stability.

2) In the case of rapidly weathering rocks

Weathering often progresses rapidly in soft rocks such as mudstone, shale, and poorly consolidated tuff that have undergone structural movement in the deformation zone, as well as in metamorphic rocks like serpentinite and hot spring residue. This is due to their inherently low shear strength combined with environmental factors such as stress release from excavation, followed by repeated cycles of drying and wetting or freezing and thawing. Consequently, collapse frequently occurs.

3) When shear strength is inherently low

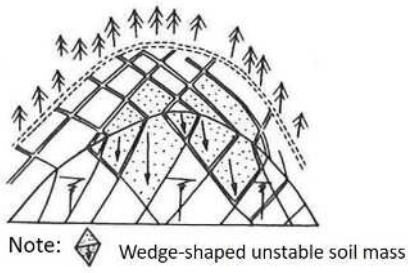
It is difficult to expect that the ground becomes harder the deeper it is below the ground surface. Therefore, failures extending deep and wide, including deep-seated strata, may occur. When such failures are anticipated, slope gradients should be evaluated through stability calculations.

4) When weathering progresses rapidly due to environmental factors such as stress release

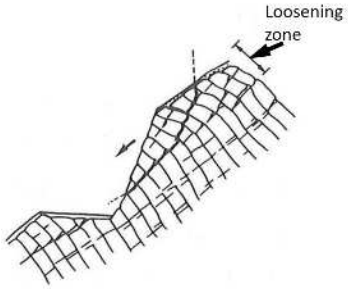
During cut construction, slopes that are initially hard and stable may become unstable over time as they weather into loose soil. To ensure a stable slope gradient that prevents collapse even as weathering progresses in the future, the slope gradient should be made gentler than that determined by standard stability analysis. Additionally, to suppress weathering as much as possible, enclosed slope protection works such as shotcrete works or concrete crib works are used in combination.

(5) In the case of rock with numerous fractures

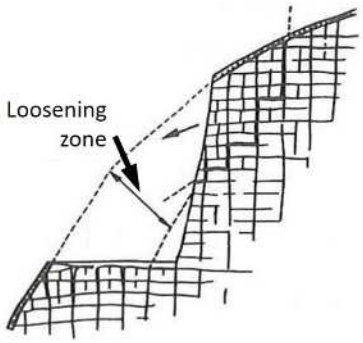
Rock masses often develop many fractures, such as joints, small faults, and thin-layer fractures formed by cooling contraction of lava or by faulting. Collapse along fractures (joints, small faults, thin layers) or geological structures within the rock mass is likely to occur (Figure 4.4.9).



(a) Collapse of a wedge-shaped unstable soil mass



(b) Collapse of the loosening zone at the surface of the supporting strata



(c) Collapse of the loosening zone at the surface of rock with developing horizontal and vertical fractures

Figure 4.4.9 Schematic Diagram of Slope Failure in Cut Slopes with Highly Fractured Rock (JRA,2009)

The slope gradient in such cases shall be determined by considering the degree of joint development, the roughness and looseness of joint surfaces, the nature of intercalated materials, the orientation of surfaces and their mutual angles, and their relationship to the direction of the cut slope.

(6) When Fractures are dip slopes

When fractures form a dip slope, cut slopes are prone to failure along the dip slope structure. The stability of cut slopes is influenced by the slope gradient and direction relative to the fracture. Therefore, the slope gradient should be determined considering the apparent fracture dip angle (Figure 4.4.10) in a section perpendicular to the slope.

$$\tan\alpha' = \cos\theta \cdot \tan\alpha$$

Here, θ: Angle between the strike of the joint and the slope direction ($\theta = 0°$ to $90°$)
α: Inclination angle perpendicular to the strike of the joint, i.e., the true inclination angle of the joint
α': Apparent inclination angle of the joint in a section perpendicular to the slope

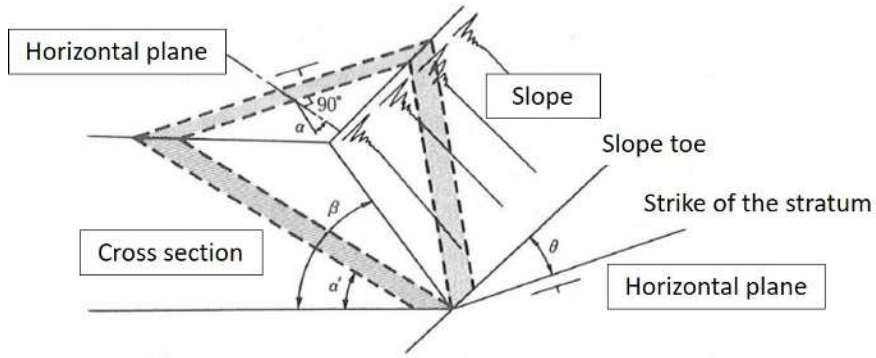
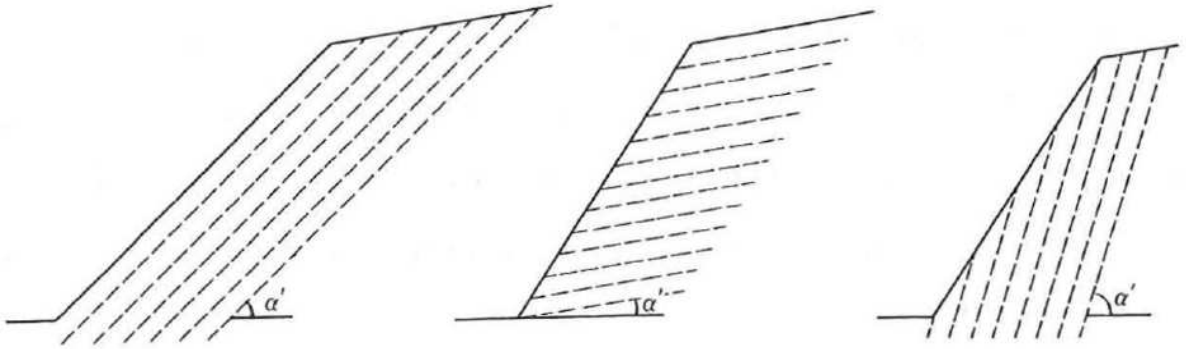


Figure 4.4.10 Relationship between Fracture Angle and Slope Angle (JRA,2009)

Slopes should generally be designed with a gradient equal to or less steep than the apparent dip angle α' of the joint (Figure 4.4.11(a)). However, as shown in Figure 4.4.11(b), in the case of a dip slope where the apparent dip angle α' of the fracture is less than 30°, the slope is not necessarily unstable unless the cut slope gradient is gentler than the apparent dip angle of the fracture (or a gradient of 1:1.8) (≈30°). Therefore, the slope gradient may be determined based on other factors (e.g., the degree of fracture development). On the other hand, in the case of a dip slope where the apparent dip angle α' of the joint exceeds 60° (Fig. 4.4.11(c)), a slope gradient of 1:0.6 (≈ 60°), which is equal to the apparent dip of the joint, is not necessarily stable.Generally, for dip slopes, it is advisable not to use a gradient steeper than 1:0.8 (≈50°) for slopes with a total vertical height of 10 meters or more.



(a) For a typical dip slope (30°＜α'＜60°)

(b) For a gently sloping dip slope (α'＜30°)

(c) For a steeply sloping dip slope (60°＜α')

Figure 4.4.11 Relationship between Cracks and Slope in Dip Slopes (JRA, 2009)

(7) When Groundwater is Abundant

The majority of slope failures are directly or indirectly influenced by groundwater. Therefore, when cutting slopes at locations with abundant seepage or high groundwater levels, depending on geological conditions, the slope may possess unstable elements. In such cases, either the slope gradient should be made gentler than the cutting slope gradient determined by standard stability analysis, or drainage measures should be implemented to create a slope condition less prone to failure.

Design underground drainage works (groundwater drainage ditches, gabion works, horizontal drainage holes) as drainage measures to collect seepage water from the slope and groundwater near the ground surface and drain it away from the slope.

(8) When a Long Slope is Unavoidable

A long slope could cause a major disaster if it collapses. Furthermore, modifying the cut (re-cutting) after excavation has progressed is disadvantageous both economically and in terms of constructability. Therefore, it is desirable to reduce the amount of cut modification beforehand through minor route shifts, etc. If a long slope is unavoidable, it is advisable to design it with greater margin compared to a standard cut. For example, with expansive rock or rapidly weathering rock, adopt a gentler slope than that determined by standard stability analysis. This accounts for secondary strength reduction due to stress release after cutting and rapid weakening from accelerated weathering, ensuring slope safety even if weakening progresses rapidly. Additionally, for the slope, wide terraces approximately 3m to 4m wide should be provided every 20m to 30m vertical height.

References

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

MLIT (2020): *Design Guidelines: Highways*, Ministry of Land, Infrastructure, Transport and Tourism (MLIT), Hokuriku Regional Development Bureau.

5. Design of Large-diameter drainage wells

5. 1 Overview of Large-diameter drainage wells

5. 1. 1 Purpose of Large-diameter drainage wells

Implemented to reduce or halt landslide activity by draining groundwater near the slip surface for reactivated slow-moving landslides meeting all of the following conditions:

1) Reactivated Slow-moving landslides triggered by increased groundwater due to rainfall, snowmelt, etc.

2) Landslides possessing an existing slip surface with a maximum depth of approximately 10m or more

3) Reactivated Slow-moving landslides with a maximum movement velocity during the active phase of approximately 1m/d or less, where slow-moving landslide activity has largely subsided

5. 1. 2 Characteristics of the Construction Method

The Japanese large-diameter drainage wells method was developed in Japan in 1955 as a deep groundwater drainage technique combining a large-diameter well shaft with drainage and discharge pipes from within the shaft. This large-diameter drainage wells were made of reinforced concrete (RC) and was developed in Niigata Prefecture based on materials such as the Manchurian well previously developed by Japanese engineers and the American radial well. In 1961, a steel large-diameter drainage well method using liner plates was developed as a uniquely Japanese technique.

For slow-moving landslides which exceed approximately 15m in slip depth, it becomes difficult to drain groundwater near the slip surface using surface drainage boreholes. Generally, multiple groundwater zones exist in reactivated slow-moving landslide areas, but removing groundwater from the zone near the slip surface is most critical. In such cases, as shown in Figure 5.1.1, digging a large-diameter drainage well within the slow-moving landslide area and installing drainage pipe from the well to the vicinity of the slip surface allows efficient collection of groundwater near the slip surface. The collected groundwater is then discharged outside the landslide area via discharge pipes.

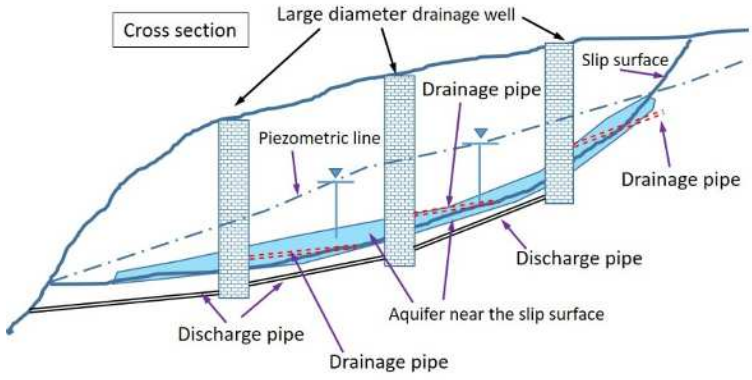


Figure 5.1.1 Schematic Diagram of Deep Groundwater Drainage and Discharge Using a Large-diameter drainage wells

Specifically, construction will be carried out according to the following procedures and specifications.

1) Drill a well with a diameter of 3 to 5 meters and a depth of 10 to 100 meters within the active layer of the slow-moving landslide.

2) Install a 30 to 100-meter-long lateral drainage pipe from within the well to collect groundwater near the slip surface into the well.

3) Install a storage tank and discharge pipe at the bottom of the well to discharge the collected groundwater outside the slow-moving landslide area.

In cases of large-scale slow-moving landslides, multiple large-diameter drainage wells may be placed at intervals of several tens of meters, with discharge pipe connecting the wells to discharge groundwater outside the slow-moving landslide area. The shaft lining material may be steel, such as liner plates (steel large-diameter drainage wells, see Figure 5.1.2), or reinforced concrete (RC large-diameter drainage wells, see Figure 5.1.3).



Figure 5.1.2 Example of a Steel large-diameter drainage well (MAFF, 2024)

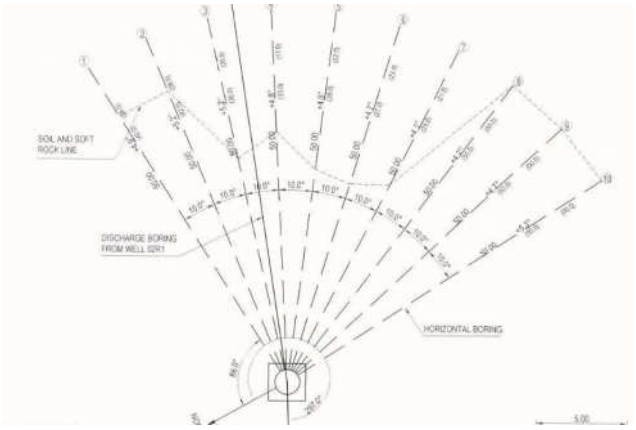


Figure 5.1.3 Example of RC Large-diameter drainage well Structure (Ito, 2020)

Figure 5.1.4(a) shows the drilling of drainage pipes from within the large-diameter drainage well. A compacted drilling machine is used to perform the work within the confined space of approximately 3.5 meters in diameter. Figure 5.1.4(b) is a layout diagram of the drainage pipes, which are arranged radially from within the large-diameter drainage well. This diagram shows a plan for ten drainage pipes, each 50 meters in length.



(a) Drilling Status



(b) Layout Diagram of Drainage Boreholes

Figure 5.1.4 Drilling Status and Unfolding Diagram of the Drainage Boreholes from Inside the Large-diameter drainage well

In Japan, this method is adopted for many reactivated slow-moving landslides, and as of 2012, approximately 11,000 large-diameter drainage wells structures had been constructed. Generally, RC large-diameter drainage wells offer superior durability and safety compared to steel ones, yet steel large-diameter drainage wells are overwhelmingly more commonly adopted in Japan. The primary reason is that RC large-diameter drainage wells incur higher material, transportation, and construction costs compared to steel ones. Additionally, the inability to install components manually makes them disadvantageous under typical mountainous ground conditions.

The slow-moving landslide suppression effect of large-diameter drainage wells can be evaluated by measuring the difference in groundwater levels before and after construction and analyzing changes in the safety factor through slope stability analysis.

5. 1. 3 Structure of the Large-diameter drainage well

Figure 5.1.5 shows the structure of a steel large-diameter drainage well. The main structure of the shaft lining consists of a liner plate and reinforcement rings, with vertical stiffeners and lateral struts installed as needed. A concrete storage tank (total height approx. 2m, depth approx. 1.5m) is installed at the bottom to temporarily store collected groundwater, which is then discharged outside the site via discharge pipe. The above-ground portion protrudes approximately 1m, featuring a canopy made of expanded metal or similar material, along with a protective fence to prevent intrusion.

RC (Reinforced Concrete) structure large-diameter drainage wells have a similar construction.

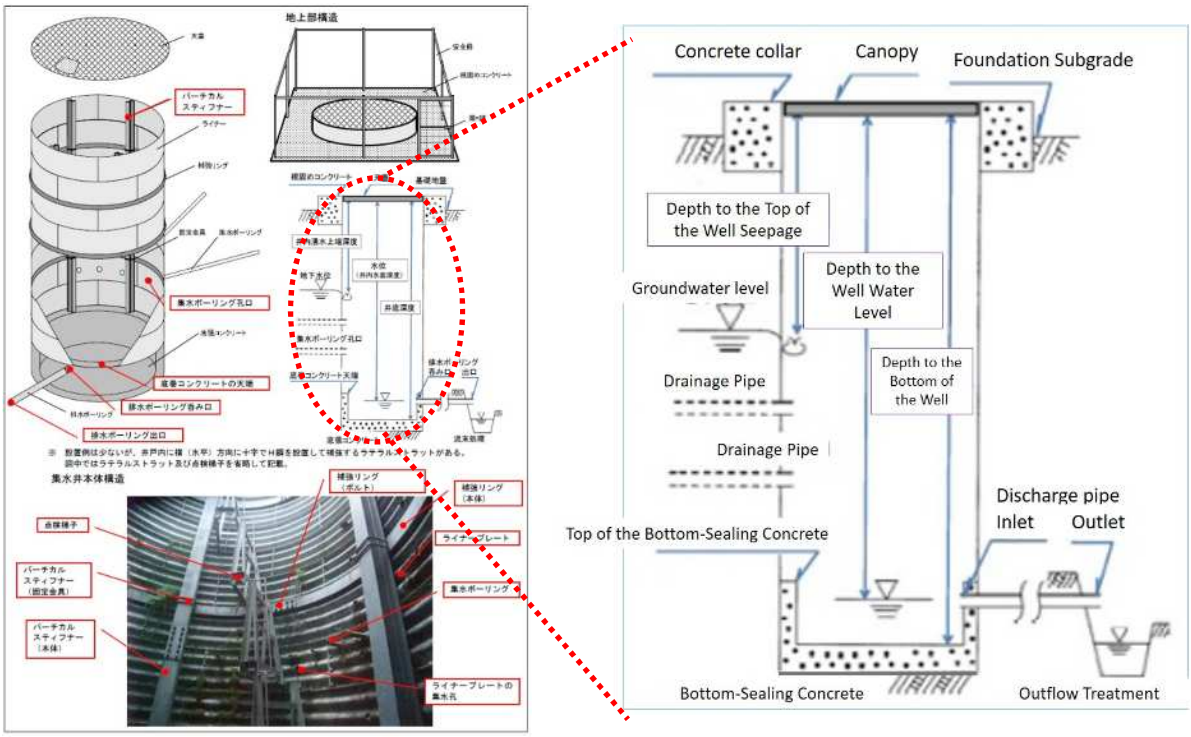


Figure 5.1.5 Structure of a Steel Large-diameter drainage well (MAFF, 2013)

References

PWRI (2007): *Technical Guidelines for Slow-moving Landslide Prevention and Commentary*, Public Works Research Institute (PWRI), Technical Note of PWRI, No. 4077.

MAFF (2013): *Guidelines for Maintaining the Functionality of Slow-moving Landslide Prevention Facilities - Mitigation Measures Edition*, Ministry of Agriculture, Forestry and Fisheries (MAFF), pp.20.

MAFF (2024): *Large-diameter Drainage Wells, Ministry of Agriculture*, Forestry and Fisheries (MAFF), Kanto Regional Forest Office, (https://www.rinya.maff.go.jp/kanto/policy/business/santi-saigai/kousyu/syusui.html, Accessed 23 Jan. 2024)

Ito, K. (2020): An Example of Reinforcement Work for a Liner Plate Catchment Well, Lecture Materials for the Reiwa Era 2 (2020) Landslide Prevention Engineer Technical Seminar (Sendai).

5. 2 Key Issues and Actions to Take in Internationalization

5. 2. 1 Countermeasure Technologies Unique to Japan

(1) Issues

Large-diameter drainage wells combining shafts with drainage/discharge pipes are a countermeasure for reactivated slow-moving landslides developed in Japan and are not widely known overseas. It is necessary to ensure that personnel in the relevant countries understand their effectiveness and usefulness.

The specialized steel and well casing materials are manufactured in Japan, making it difficult to procure equivalent materials overseas. Furthermore, the boring machines used to construct the drainage and discharge holes from within the well were also developed in Japan as compact equipment capable of operating within the confined space of the well shaft. Consequently, when constructing large-diameter drainage wells overseas, these materials and machinery must be procured from Japan.

(2) Explanation

In Japan, the large-diameter drainage well method is the primary technique for groundwater removal in reactivated slow-moving landslides and has gained trust as a method to stabilize landslide activity.

The large-diameter drainage well method is optimal for deep groundwater removal because: 1) It is installed to penetrate multiple groundwater zones within the slow-moving landslide area, enabling groundwater level reduction through the well shaft itself; and 2) By installing drainage pipes from within the shaft, it can collect groundwater over a wider area than the shaft alone, allowing efficient groundwater collection even when the slip surface is deep.

Figure 4.2.1 shows the trend in the number of large-diameter drainage wells constructed in Japan (JASDiM, 2016). Although the number of constructions has decreased in recent years after peaking around 2000, the cumulative total up to 2012 was approximately 8,700 steel wells and approximately 2,300 reinforced concrete (RC) wells, totaling about 11,000 wells.

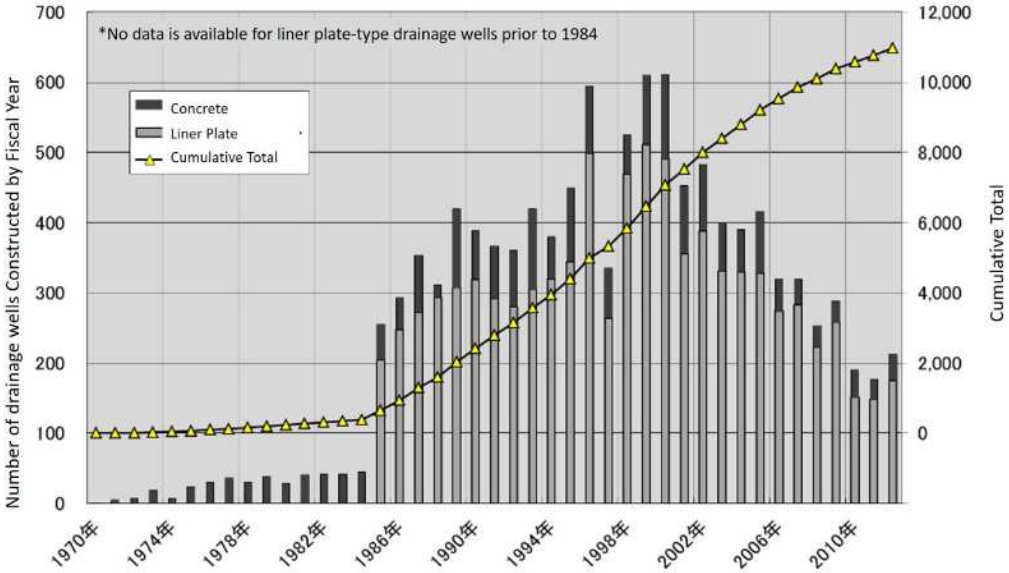


Figure 5.2.1: Trends in the Number of Construction Projects for Water Large-diameter drainage wells in Japan (JASDiM, 2016)

Large-diameter drainage wells have over 60 years of proven results in Japan's slow-moving landslide countermeasure projects and remain a primary slow-moving landslide control method widely implemented today. This track record demonstrates the effectiveness of this technique. However, drainage wells themselves are not a uniquely Japanese countermeasure. In Italy, small-diameter drainage wells (pozzi derenati) with diameters of about 1 to 2 meters have been used since at least around 1990 as a countermeasure for reactivated slow-moving landslides. These drainage wells consist of a series of permeable shafts placed continuously at intervals of several meters, with the bottoms of the shafts connected by discharge pipes. Large-diameter reinforced concrete pozzi drenanti also exist.

(3) Actions to Take

Demonstrate the track record in Japan and the effectiveness of this method as a countermeasure against reactivated slow-moving landslides to help engineers in developing countries understand its necessity. Utilize this book for that purpose. Consideration must also be given to the cost of shipping necessary materials and construction machinery from Japan.

References

Fuse, H., Yagi, Y. (1974): On the Development History of the Radial Well and its Executive Example in Niigata Prefecture, *J. of the Jpn. Landslide Soc.*, Vol.11, No.2, pp.15-20.

JASDiM (2016): *Slope Protection Work Maintenance Management Implementation Guidelines*, Japan Association for Slope Disaster Management (JASDiM), 920p.

5. 2. 2 Different Design Specifications for Steel Large-diameter drainage wells

(1) Issues

For steel large-diameter drainage wells, which are the mainstream solution, two design specifications exist, differing based on the governing agency responsible for the slow-moving landslide site in question. The structure of the large-diameter drainage well, the components used, and the structural calculation methods differ between the two specifications. Specification B (Forestry Agency specification) fundamentally relies on a combination of liner plates and reinforcement rings, whereas Specification A (MLIT specification) additionally assumes the use of vertical stiffeners and lateral struts. However, structural calculation formulas for vertical stiffeners and similar components do not exist. The design methods for liner plates and reinforcement rings, as well as the methods for calculating earth pressure, also differ between the two specifications.

(2) Explanation

1) Main Differences Between the Two Specifications

While logical explanations are provided in subsequent sections, the main differences between Japan's two specifications are summarized in Table 5.2.1. These differences can be further condensed into the following four points:

a) Whether the standard structure is a fully penetrating well or a partially penetrating well

b) Differences in the earth pressure system

c) Differences in how the shaft lining elliptical deformation amount δ is specified (steel structures only)

d) Whether vertical stiffeners or lateral struts are adopted for structural members (steel structures only)

Table 5.2.1 Main Differences Between Two Design Specifications in Japan (Steel large-diameter drainage wells)

Category	Specification A	Specification B
Basic Concept	Minimizing damage to drainage wells as public facilities →Partially penetrating wells are standard.	Structure expected to provide maximum effectiveness as slow-moving landslide countermeasures →Fully penetrating wells are standard.
Well Shaft Structure (Same specifications for RC construction)	●Stronger structure to enhance resistance to external forces like earth pressure during slow-moving landslide movement ●In addition to liner plates and reinforcement rings, incorporates vertical stiffeners and lateral struts as standard.	●Structure designed so that deformation due to slow-moving landslides does not impair the drainage/discharge function of the well, as no restraint function is expected from the large-diameter drainage well itself. ● Since the structure consists only of liner plates and reinforcement rings, the drainage and discharge functions are maintained even if the shaft lining shifts near the slip surface due to slow-moving landslide movement.
Differences in Design Methods (For RC construction, only the earth pressure method differs in specification)	● Earth Pressure Method: Empirical earth pressure method (equivalent to clay soil) assumed as the non-seismic lateral pressure during Japanese caisson foundation design ●Partial Earth Pressure: Considers approximately 100 kN/m² partial earth pressure in addition to the earth pressure from the earth pressure method. ● Shaft Lining Elliptical Deformation δ_0: Assumed to be 1% of the large-diameter drainage well radius r (1.75 cm) → Consequently, for the same earth pressure, thinner steel materials (liner plate, reinforcement ring) than in Specification B are sufficient.	●Earth pressure formula: Rankine earth pressure formula or Terzaghi earth pressure formula, considering soil strength constants ●Partial earth pressure: For steel structures, partial earth pressure is assumed to be zero. For RC structures, partial earth pressure of approximately 100 kN/m² is considered in addition to the earth pressure from the earth pressure formula. ●Elliptical deformation amount δ_0 of the shaft lining: Assumed to be 4–10 cm as the over-excavation allowance during large-diameter drainage well construction.
Differences in Structural Calculation Approach	●The earth pressure (external force) is assumed to be the worst-case scenario (clay soil) regardless of actual ground conditions, applying a larger earth pressure (external force). ●In structural design, setting the elliptical deformation to a small value reduces the required sectional force, resulting in a design where the required sectional performance for the liner plate and reinforcement ring is smaller.	●For the external soil pressure, values corresponding to the soil properties (γ, c, φ) are calculated, taking the actual ground conditions into account. ●In structural design, the maximum elliptical deformation is set generously to account for the equivalent overburden, increasing the required sectional force. This results in a design where the required sectional performance for liner plates and reinforcement rings is larger.

※Fully penetrating well = Well penetrating the slip surface; Partially penetrating well = Well not reaching the slip surface

2) The Role and Drawbacks of Vertical Stiffeners and Lateral Struts

Liner plates and reinforcement rings are structurally calculated to determine their buckling resistance and earth pressure resistance, establishing their specifications and placement. However, vertical stiffeners and lateral struts, while structural components of the large-diameter drainage well, lack established structural calculation methods. Steel specifications and spacing are determined by long-standing convention without structural calculations. While these two members are intended to reinforce the shaft, vertical stiffeners increase the shaft's resistance to external forces only in the direction they are installed. They have no effect in other directions, and installing only about four provides limited reinforcement for the entire shaft. Lateral struts increase the buckling resistance in the axial force direction at the depth where they are installed, but they do not contribute at other depths. Their overall reinforcement effect on the shaft is also limited. The primary effect of these two members is to foster a psychological sense of security during construction.

On the other hand, when a large-diameter drainage well undergoes shear deformation due to shallow slow-moving landslides, a structure consisting solely of liner plates and reinforcement rings will only cause lateral displacement of the liner plates at specific depths. However, if vertical stiffeners are employed, there is a risk that

the impact could extend to the entire depth. Even in shallow slow-moving landslides, vertical stiffeners or lateral struts cannot counteract the earth pressure associated with slow-moving landslide movement.

Despite the fact that most large-diameter drainage wells under Specification B are fully penetrating wells installed below the slip surface and are thus strongly affected by slow-moving landslide movement, no major problems have arisen from not using vertical stiffeners or similar devices. Specification B also used vertical stiffeners until around the early 1980s, but their use ceased due to frequent damage to the entire well shaft caused by slow-moving landslide movement.

(3) Actions to Take

Vertical stiffeners and lateral struts are not recommended because they increase material and construction costs and have the adverse effect of propagating partial shaft lining deformation caused by shallow slow-moving landslide movement to the entire shaft depth. It is difficult to explain in countries other than Japan why it is necessary to install steel materials that are not subject to structural calculations. For steel large-diameter drainage wells, a structure that counters earth pressure using only liner plates and reinforcement rings is recommended. Furthermore, specifications (including design methods) that can provide logical explanations acceptable to engineers in those countries must be selected for overseas deployment. Details are described in the subsequent section on responses to individual specifications.

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5. 2. 3 Differences in Excavation Depth Between Two Specifications

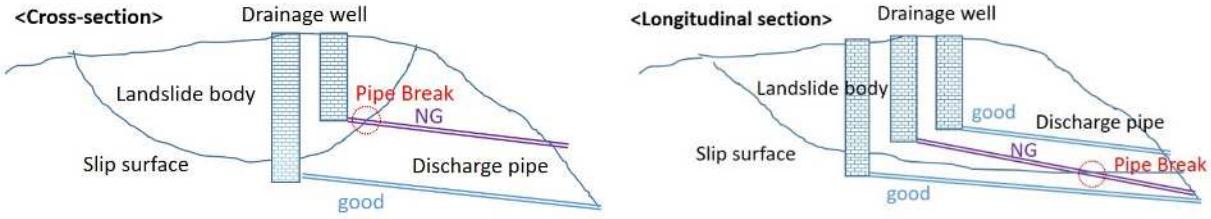
(1) Issue

Both Japanese specifications (Specification A and Specification B) assume penetration of approximately 2 to 3 meters into the bedrock below the slip surface for stationary slow-moving landslides. They also share the principle of placing the bottom above the slip surface for active landslides. Typical reactivated slow-moving landslides often exhibit some ongoing activity or alternate between movement and stasis. For such landslides, Specification A generally does not require penetration as per the principle, while Specification B typically does require penetration. Thus, Specifications A and B differ in their excavation depth requirements. Specification B effectively allows for some deformation of the shaft lining due to slow-moving landslide activity.

(2) Explanation

By installing the well shaft deeper than the slip surface, it is possible to penetrate all groundwater zones—including those above and below the slip surface and those within the moving layer—thereby reducing the pressure head of each groundwater zone at that location to nearly zero, allowing the well to fully demonstrate its water collection function as a fully penetrating well. Since the discharge pipe can be positioned entirely within the immovable layer, the risk of damage to the discharge pipe can be minimized. In the case of steel large-diameter drainage wells, even if the shaft lining is destroyed by lateral shear failure at the slip plane depth due to slow-moving landslide activity, the drainage and discharge functions are largely unaffected. Conversely, placing the bottom within the mobile layer results in a partially penetrating well, reducing the drainage function of the shaft itself. Furthermore, placing the discharge pipe entirely within the mobile layer increases the risk of damage, as the mobile layer deforms during landslide activity, potentially creating step cracks or open cracks. Therefore, focusing on maximizing effectiveness as a slow-moving landslide countermeasure, the method of placing the bottom below the slip surface is superior. However, when vertical stiffeners are used with steel large-diameter drainage wells or in reinforced RC large-diameter drainage wells, placing the well shaft below the slip surface may increase the risk of deformation or damage to the shaft lining due to landslide movement.

When slow-moving landslide activity is active and there is a possibility that the shaft lining could be severely damaged by landslide activity, the bottom should be set in the mobile layer (partially penetrating well). The discharge pipe will be installed within the mobile layer, but it must be positioned to prevent damage from deformation of the moving soil mass. Designing the discharge pipe to penetrate the slip surface should be avoided, as this could cause fracture due to slow-moving landslide movement and supply collected groundwater to the slip surface. However, positioning it without intersecting the slip surface is often difficult.



a) Relationship in the cross-sectional plane **b) Relationship in the longitudinal section**

Figure 5.2.2 Quality of Relationship Between Discharge Pipe and Slip Surface

(3) Actions to Take

Steel large-diameter drainage wells are preferable for overseas projects due to transportation costs and constructability.

For particularly slow landslide activity with maximum annual displacement of several centimeters, fully penetrating wells are recommended due to their drainage efficiency and ease of discharge pipe placement. For active landslides exceeding this annual displacement, design as partially penetrating wells might be recommended. However, if discharge pipe cannot avoid penetrating the slip surface, design as fully penetrating wells is recommended.

While RC large-diameter drainage wells generally offer excellent durability, designing them as partially penetrating wells is preferable to avoid damage from landslide movement that could occur with fully penetrating wells. Their adoption requires careful consideration, given the higher transportation costs from Japan compared to steel alternatives and the limited number of Japanese companies with proven construction experience.

5. 2. 4 Design Calculations for Steel Large-diameter drainage wells Differing by Specification

(1) Issues

In Japan, the design methodologies for Specification A and Specification B differ in the following aspects:

- The earth pressure method differs; both specifications utilize a uniquely Japanese earth pressure method which is not used overseas.
- The design methods for steel large-diameter drainage wells partially differ.

(2) Explanation

1) Explanation of Earth Pressure Formulas

a) Issues with Rankine's and Coulomb's Earth Pressure Formulas

Rankine's earth pressure formula (1857) and Coulomb's earth pressure formula (1773) are well-known earth pressure formulas both in Japan and internationally. When limited to horizontal earth pressure acting on a vertical wall surface, Rankine's formula coincides with Coulomb's formula when the friction angle δ on the wall surface is set to $\delta = 0°$. These soil pressure equations pertain to unconsolidated soil pressure and are theories for relatively shallow depths. Large-diameter drainage wells typically operate at depths of 20 to 50 meters, with some reaching over 100 meters. These soil pressure equations are not designed for such great depths. Therefore, using them may result in calculated soil pressures at greater depths being larger than actual values.

Terzaghi (1943) and Terzaghi and Peck (1948) compared Rankine soil pressure (1857) with measured values. They demonstrated that vertical arch action from the surrounding soil acts on the soil pressure exerted on a vertical wall surface. Consequently, soil pressure decreases near the bottom of the wall surface, leading to a larger discrepancy with Rankine soil pressure. Therefore, the earth pressure calculated using Rankine's formula tends to be larger than measured values at greater depths. Since the target soil often consists primarily of weathered rock at depth, the discrepancy with actual conditions is expected to increase further. On the other hand, even when applying a Rankine formula adapted for multi-layer structures, the tendency to calculate large earth pressures at great depths remains the same. Increasing the thickness of a single layer within the multi-layer structure further intensifies this tendency. Furthermore, due to the effect of cohesion c, negative earth pressure may sometimes be calculated even in intermediate or deep layers.

b) Problems with the Terzaghi Earth Pressure Formula Used in Japan

In Japan, the Terzaghi earth pressure formula incorporating arch action has been used since around 1980 (Showa 55) and is still adopted in the Technical Standards for Specification B. This formula was not proposed by Terzaghi himself, but was independently derived by researchers at Japan's Ministry of Construction in 1980. Consequently, this formula is not known internationally. Lacking theoretical justification, its use overseas is difficult.

c) Issues with Empirical Earth Pressure Methods

Figure 5.2.3 shows the method for calculating earth pressure acting on temporary earth retaining structures in construction projects in Japan.

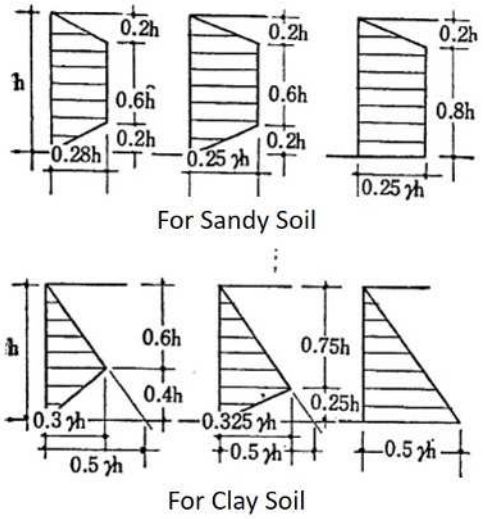


Figure 5.2.3 Japan Society of Civil Engineers Standard Earth Pressure Calculation Method (Fujimori and Uchida, 1974)

Such empirical formulas determine the depth at which the earth pressure distribution pattern changes as a ratio of the excavation depth h, and they modify the earth pressure coefficient and distribution pattern according to the geology of the target ground. These are empirical formulas nearly identical to those proposed in foreign literature. Terzaghi and Peck (1948) also proposed similar empirical formulas.

On the other hand, the Japanese Road Bridge Specifications - Substructure Edition (JRA, 1994) adopts static earth pressure as the non-seismic load on the sidewalls of caisson foundations after completion. It states: “The static earth pressure coefficient shall be 0.5, regardless of whether the soil is sandy or cohesive. The distribution of static earth pressure strength shall be considered triangular up to a depth of 15m, and at depths greater than 15m, it shall be considered equal to the earth pressure strength at a depth of 15m.” This method was adopted in Specification A. This standard in the Road Bridge Specifications concerning caisson foundations targets soils such as sandy soil and cohesive soil, and does not consider weathered rock, which is common in slow-moving landslide areas. Caisson foundations are large, highly rigid reinforced concrete piles. Since they typically do not fail due to post-construction earth pressure, it is considered that a large earth pressure is assumed as an external force on the safety side.

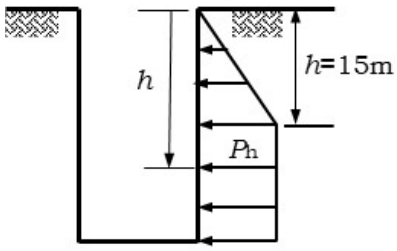


Figure 5.2.4 Soil Pressure Distribution Adopted by Japan's Ministry of Land, Infrastructure, Transport and Tourism (PWRI, 2008)

Conventional empirical earth pressure formulas employ soil pressure coefficients specific to soil types. However, this method assumes a constant soil pressure coefficient (0.5), resulting in a uniform earth pressure value regardless of geological conditions. This implies identical earth pressures for slow-moving landslides dominated by colluvial deposits and those dominated by weathered rock, introducing a logical contradiction.

However, no other existing soil pressure formulas exist that can accurately calculate appropriate soil pressure for either colluvial deposits or weathered rock. Considering its widespread adoption in Japan, applying the soil pressure distribution shown in Figure 4.2.4 for complex ground conditions is a viable proposal.

d) Issues Concerning the Consideration of Partial Earth Pressure

Specification A states that partial earth pressure should be considered in the design of steel large-diameter drainage wells when it is anticipated. However, in practice, structural calculation formulas accounting for partial earth pressure in two or four directions are not utilized. The *Liner Plate Design and Construction Manual* (TACPL, 2009) states that for large-diameter drainage wells installed within or near slow-moving landslide areas where partial earth pressure is anticipated, an empirical value of approximately 100 to 200 kN/m² should be assumed to act along the entire depth of the well. However, in practice, the structural calculation formula for partial earth pressure from four directions described here is not utilized. Specification B generally does not consider partial earth pressure (except for RC large-diameter drainage wells). Thus, it is common practice not to consider partial earth pressure for steel large-diameter drainage wells.

On the other hand, the manual by HCA (2010), the only design manual for reinforced concrete large-diameter drainage wells, explicitly states that partial earth pressure of 100 kN/m² must be considered. Since the design formula adopted assumes the presence of partial earth pressure, setting it to zero renders the design calculations impossible. Therefore, partial earth pressure is essential in the design of RC large-diameter drainage wells.

2) Explanation of Design Calculations for Steel Large-diameter drainage wells

Technical guidelines and standards documents concerning Specification A do not describe the structural calculation method for steel large-diameter drainage wells. The detailed structural calculation method for steel large-diameter drainage wells is believed to have depended on steel manufacturers since the time of development. Therefore, the structural calculation method is identical for both Specification A and Specification B. The only difference in design calculations between Specification A and Specification B lies in how the elliptical deformation amount is specified when calculating the maximum bending moment of the shaft lining. Equation 5.2.1 is the

structural calculation formula for determining the section forces (maximum bending moment M_{max}) based on the elliptical deformation of the shaft lining. The original source of this equation is the formula related to buckling of a ring, as presented by Timoshenko and Gere (1961).

$$M_{max} = \frac{PU_0R}{1 - \frac{P}{P_{cr}}}$$

Here, M_{max}: Maximum bending moment per meter of depth (kN·m/m)

P: Earth pressure per meter of depth (kN/m^2) ・・・5.2.1

R : Radius of the large-diameter drainage well (m)

U_0 : Elliptical deformation (m)

P_{cr}: Buckling limit earth pressure per 1 m depth ($=3EI/R^3$)(kN/m^2)

EI: Composite bending stiffness of the liner plate and reinforcement ring per 1 m depth (kN·m^2/m)

Specification B explicitly states in official technical standards that the clearance for the excavation margin during construction (see Figure 5.2.5) is used as the basis for setting the elliptical deformation amount. This means the excavation margin during construction is considered as the maximum deformation amount when subjected to earth pressure. In contrast, Specification A sets the elliptical deformation amount as "1% of the large-diameter drainage well radius *r*." This value was arbitrarily set during preliminary calculations when comparing design methods in a 1980 study by a national research institute and lacks a logical basis. While this specification is not documented in official technical standards for Specification A, it is still used in practical design calculations today.

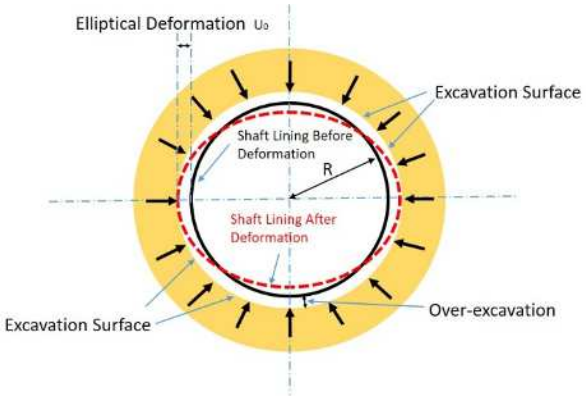


Figure 5.2.5 Concept of Elliptical Deformation of the Shaft Lining in Specification B

For Specification B, the elliptical deformation amount δ is set within the range of 4 to 10 cm, considering the excess excavation during construction, with 5 cm being the standard value adopted in most cases. For a typical 1.75 m radius large-diameter drainage well, specification A yields an elliptical deformation amount δ of 1.75 cm. This is 2.8 times smaller than the typical value of 5 cm for specification B. Consequently, the calculated maximum bending moment on the steel members for specification A is 2.8 times smaller than for specification B, allowing the permissible stress to be satisfied with a thinner liner plate. The required reinforcement ring spacing is also wider for Specification A, reducing the necessary quantity. In normal construction, achieving a small excavation margin of about 1.75 cm for the large-diameter drainage well is physically difficult when considering the connection structure between the liner plate and reinforcement rings, as well as the placement of fixing bolts.

Therefore, Specification B, which considers a practical excavation margin, is logically easier to understand.

(3) Actions to Take

1) Appropriate Approach to Earth Pressure Formulas

Japan's Specification A empirical earth pressure formula lacks theoretical basis. However, considering its extensive adoption in Japan, it could be proposed as an empirical formula for complex ground conditions. Since multiple empirical formulas exist internationally, it is advisable to adopt the optimal empirical formula after comparing them. On the other hand, Rankine's soil pressure formula is globally recognized, and multiple related papers demonstrate its theoretical basis, making it readily accepted for use overseas. However, it tends to calculate excessive soil pressure at deep excavation depths and has the issue that the soil pressure at the excavation bottom—where actual soil pressure is zero due to arch action—is calculated as the maximum. The Terzaghi soil pressure formula used in Japan is specific to Japan, making its application overseas difficult. Therefore, it is considered preferable to list both globally recognized soil pressure formulas, such as Rankin's, and the empirical soil pressure formula of Specification A, allowing designers to choose.

Regarding partial earth pressure, as explained later, it is not considered because Specification B is adopted for setting the elliptical deformation amount. However, when designing RC large-diameter drainage wells, partial earth pressure is essential, so it must be set similarly to Japan.

2) Design Calculations for Steel Large-diameter drainage well Structures

The elliptical deformation amount δ is recommended to adopt Japanese specification B, which allows for logical explanation, and it is desirable to set it within the range of over-excavation during construction (4–10 cm).

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5. 2. 5 Validity of Structural Calculations for RC Large-diameter drainage wells

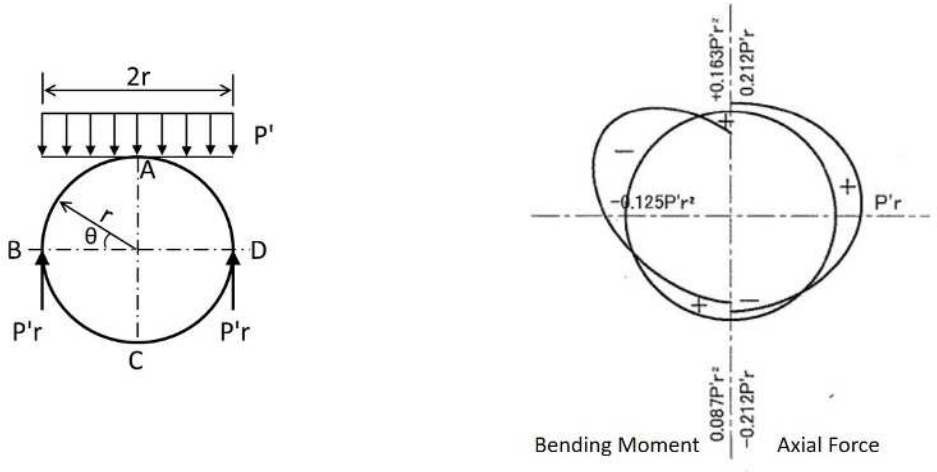
(1) Issues

The structural calculation formulae used for RC large-diameter drainage wells in Japan cannot be verified for validity using general structural mechanics. Furthermore, no formulae exist for calculating maximum section forces such as bending moments or axial forces. Furthermore, the method for setting the partial earth pressure—used as the primary external load in structural calculations—differs from the original source proposing this method in terms of the load occurrence conditions. It also differs from the partial earth pressure setting methods adopted in other technical guidelines concerning deep circular shafts in Japan.

(2) Explanation

1) Explanation Regarding Structural Calculation Formulas

As of November 2025, the primary design manual for reinforced concrete large-diameter drainage wells is the *Reinforced Concrete large-diameter drainage wells: Design and Construction Manual* (HCA, 2010).



(a) Conditions of loads acting on the shaft lining

(b) Distribution of section forces occurring in the shaft lining

Figure 5.2.6 Distribution of partial earth pressure and section forces acting on the shaft lining (HCA, 2010)

Figure 5.2.1(b) shows the axial force at point A as 0.212 *P'r*, indicating a finite value. However, under the loading conditions in Figure 5.2.1(a), where no lateral external forces are present, the axial force at point A is always zero according to structural mechanics. Furthermore, assuming the bending moment at point B in Figure 5.2.1(a) is M_0, the bending moment M_θ between points B-A-D is expressed by Equation (1) according to general structural mechanics.

$$M_\theta = M_0 + P'r(r - r\cos\theta) - \frac{1}{2}P'(r - r\cos\theta)(r - r\cos\theta)$$

$$= M_0 + P'r^2(1 - \cos\theta) - \frac{1}{2}P'r^2(1 - \cos\theta)^2$$

$$= M_0 + \tfrac{1}{2}P'r^2(1 - \cos^2\theta)$$

$$= M_0 + \frac{1}{2}P'r^2\sin^2\theta \qquad \cdots (1)$$

In Figure 5.2.6(b), the bending moment M_0 at point B is $M_0 = -0.125P'r^2$. Substituting this into Equation (1) to find the bending moment M_{90} at point A ($\theta = 90°$), we obtain $M_{90} = 0.375\ P'r^2$. This value differs from the value at point A in Figure 5.2.6(b). Thus, the section force distribution in Figure 5.2.1(b) cannot be explained by general structural mechanics. Furthermore, assuming the section forces in Figure 5.2.6(b) are correct, the position $\theta \approx 40°$ where the bending moment is maximum (relative to point B) should also be verified. However, the formula for calculating the section forces at this position is unknown.

2) Explanation of Partial Earth Pressure

Since slow-moving landslide thrust forces are generally enormous, reinforced concrete large-diameter drainage wells cannot withstand such forces. Therefore, it is necessary to consider partial earth pressure unrelated to slow-moving landslide thrust forces. HCA (2010) assumed a static earth pressure of 140 kN/m² by adding the above-ground load to the normal static earth pressure. They then set the partial earth pressure to 100 kN/m² (71% of the static earth pressure), which is greater than half of the static pressure. According to Enokida's (2026) preliminary calculations, using the conventional structural calculation formula results in a required number of 5 bars each for both tension and compression steel. However, applying the new structural calculation formula (Equation 2) requires 7 bars for each type. However, if an partial earth pressure of 68 kN/m² is adopted—half of the static earth pressure of 135 kN/m² (ignoring the superimposed load)—then even using the new structural calculation formula, only 4 bars each of tension and compression steel are required.

If no major problems have arisen with the number of reinforcing bars calculated using the previous structural formula, it is advisable to adopt the new structural formula after reviewing the setting values for partial earth pressure.

In the design of deep circular shafts associated with various tunnels, uniaxial earth pressure from two directions is considered in addition to uniform earth pressure from all directions. Table 5.2.2 shows the approach to earth pressure described in technical guidelines related to the design of deep circular shafts. Uniaxial earth pressure is considered at 5 to 20% of the normal earth pressure. Iwanami (2003MS), in analyzing existing design cases for deep circular shafts, presents seven representative examples (inner diameters 14–76 m). These show that many cases adopt lateral pressure equivalent to 20% of the non-seismic earth pressure. Adopting partial earth pressure of 27 kN/m² (20% of the static earth pressure of 135 kN/m²) results in a requirement for only one bar each of tension and compression steel, even when using the new structural calculation formula.

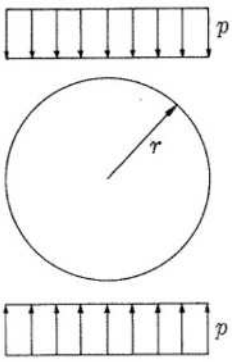
Table 5.2.2: Example Standard Values for Earth Pressure in Deep Circular Shafts

Standards	*Guidelines for Design and Construction of Deep Earth Retaining Structures* (ACTEC, 1994)	*Guidelines for Investigation, Design, and Construction of Deep Shafts and Tunnels for Underground Power Transmission* (JTA, 1982)	*Design and Construction of Underground Structures for Electric Power Facilities* (JEPOC, 1986)
Earth Pressure	Static earth pressure is adopted. **<Sandy Soil>** K0=1-sinφ (Jaky's empirical formula) **＜Cohesive Soil＞** 1) $N \geqq 8$: K0=0.5 2) $8 > N \geqq 4$: K0=0.6 3) $4 > N \geqq 2$: K0=0.7 4) $2 > N$: K0=0.8	Use a value close to the static earth pressure. 1) Gravel with $N < 50$ or cohesive soil with $N < 10$: K0 = 0.5 2) Gravel with $N \geq 50$: K0 = 0.3 3) Cohesive soil, mudstone, consolidated silt, or very stiff clay with $N \geq 10$: P = 100–200 kN/m² (constant value regardless of depth).	Use the static earth pressure. 1) Generally, K0 = 0.5 2) Sandy soil: K0 = 0.4–0.6 3) Cohesive soil: K0 = 0.5 to 1.0
Partial Pressure	In circular shaft design, generally consider about 10 to 20% of the static earth pressure.	For circular shafts, consider about 5 to 10% of the lateral pressure.	For underground fuel storage tanks, consider about 10 to 20% of the static earth pressure.

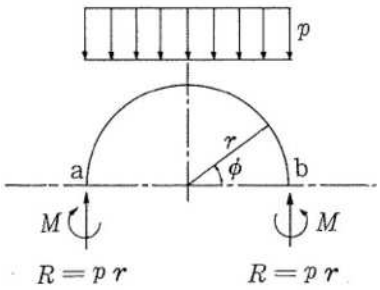
(3) Actions to Take

1) Response Regarding Structural Calculation Formulas

Recent design calculation examples for caisson foundations (JPTA, 2019) show that stress calculations for uniaxial eccentric loading consider tensile stresses on both the outer and inner surfaces as thick-walled rings. Stress calculations for thin-walled rings, as shown in Figure 5.2.6, are not performed. However, structural calculation formulas for thick-walled rings are complex, making structural calculation formulas for thin-walled rings practical, as has been the case previously. The Nakamura and Shiroishi (1980) proposed equations for the load conditions shown in Figure 5.2.3 as structural calculation formulas for shafts lining (thin-walled rings) subjected to partial earth pressure. As shown in Figure 5.2.7, when partial earth pressure acts over the entire depth of the shaft, partial earth pressure also occurs from the opposite side as a reaction force.



(a) Partial earth pressure acting on the shaft lining



(b) Conditions for calculating the cross-sectional forces on the shaft lining subjected to Partial earth pressure

Figure 5.2.7 Load conditions for a shaft lining subjected to partial earth pressure in the direction of slow-moving landslide movement (Nakamura and Shiroishi, 1980)

The calculation formula for bending moment M_φ under the load conditions shown in Figure 5.2.7 is given by Equation (2) (Nakamura and Shiroishi, 1980).

$$M_{\phi} = \frac{pr^2}{2}\left(\sin^2\phi - \frac{1}{2}\right) \quad \cdots (2)$$

Applying Equation (2) to the structural calculations for reinforced concrete large-diameter drainage wells is the most practical countermeasure. Using Equation (2) yields slightly larger section forces than the current structural calculation formula, resulting in a slight increase in the steel area required for the same partial earth pressure. If no problems have arisen with the conventional steel area, one possible approach is to reassess the partial earth pressure values and set them to smaller values than currently used.

2) Response Regarding Partial Earth Pressure

As mentioned earlier, the partial earth pressure is currently assumed to be 100 kN/m² (71% of the static earth pressure), which is greater than half (50%) of the static earth pressure of 140 kN/m², taking into account the surface load (10 kN/m²). Even this partial earth pressure of 50% of the static pressure is significantly larger than the standard values for earth pressure in other deep circular shafts (typically around 20% of the static pressure), representing an excessively high setting with multiple safety factors applied. Since the soil pressure calculation for the steel large-diameter drainage well does not consider the above-ground load, adopting partial earth pressure of 68 kN/m²—half of the static earth pressure of 135 kN/m² (ignoring the above-ground load)—is considered a realistic improvement measure.

References

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JEPOC (1986): *Design and Construction of Underground Structures for Electric Power Facilities*, Electric Power Civil Engineering Association (JEPOC), Sankaido Publishin, pp.573-593.

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ACTEC (1994): *Guidelines for Design and Construction of Deep Earth Retaining Structures (Draft)*, Advanced Construction Technology Center (ACTEC), pp.33-42.

5. 3 Design Considerations

5. 3. 1 Pre-evaluation Method for Groundwater Level Lowering Effect

In Japanese Specification A, a pre-evaluation value of 5m for the groundwater level lowering height when installing a large-diameter drainage well is widely used. A note in the official standard document issued in 1976 states: “For slow-moving landslides approximately 20m thick, the following values shall be adopted as standard. For slow-moving landslides larger than this scale, separate consideration shall be given to determine the values.” Recent official standards include the phrase “when appropriately placed within the slow-moving landslide area.” However, in practice, it remains common to assume a 5m drawdown even when the layer thickness significantly exceeds 20m or the number of wells is clearly insufficient. Since the basis for determining the 5m value was not explicitly stated in the standards, most engineers today, over 45 years later, are unaware of its rationale.

In contrast, Specification B recommends estimating the groundwater level drawdown corresponding to the well arrangement using some method. Often, the distribution of groundwater level drawdown is predicted using group well theory, which applies classical hydraulic formulas.

Since the extent of groundwater level decline caused by large-diameter drainage wells varies depending on the local groundwater level height and distribution, the permeability coefficient of the strata, and the conditions of the aquifer, the method of fixing the groundwater level decline to 5m, frequently used in Specification A, lacks a logical basis.

As recommended in Specification B, it is appropriate to analyze the groundwater level drawdown corresponding to the arrangement of the large-diameter drainage well structure by some method. Advanced groundwater seepage analysis methods such as the finite element method and finite difference method are also effective. However, their use requires caution due to the extremely poor accuracy of forward analysis without reference values and the difficulty for non-numerical analysis specialists to verify the analysis model. The application of classical hydraulic formulas, such as group well theory, is suitable for practical use as it involves fewer analysis parameters.

The large-diameter drainage well method primarily involves collecting groundwater within the well shaft via drainage pipes and discharging it through discharge pipe. However, since the well shaft with drainage holes also penetrates multiple groundwater zones at its installation site, it possesses a certain collection effect and can be expected to lower the groundwater level around the large-diameter drainage well. When estimating groundwater level reduction distribution using the group well theory under Specification B, the concept that “groundwater level reduction distribution is evaluated as the collection function of the well body itself, with collection from the drainage pipe reinforcing the well body's collection function” is introduced.

Furthermore, the actual groundwater level reduction effect must be verified post-construction through long-term groundwater level monitoring (including self-recording and manual measurements). This involves assessing not only the reduction in normal groundwater levels but also the reduction in peak levels during events such as torrential rainfall.

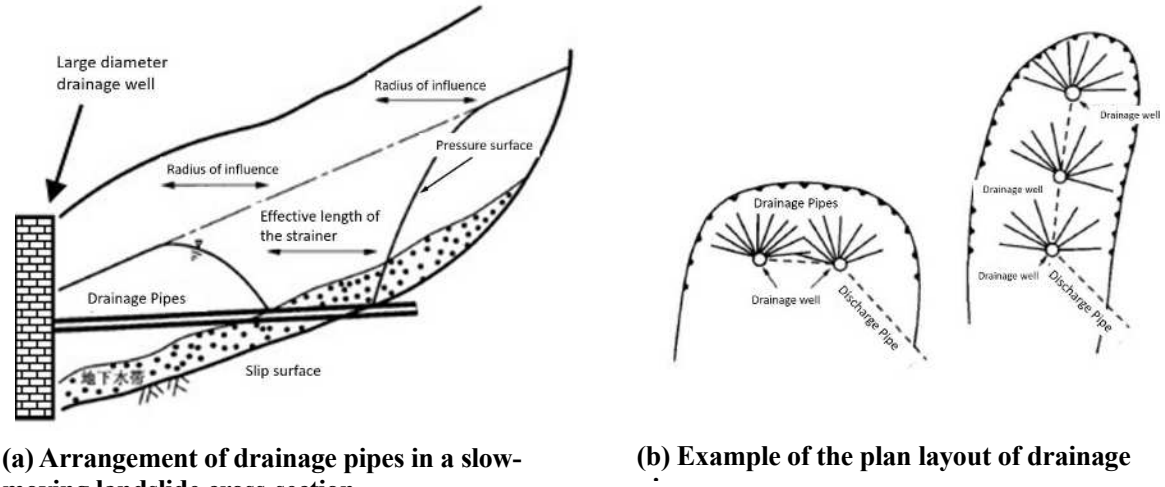
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MOC (1976): *Ministry of Construction (MOC), Technical Criteria for River and Erosion Control Works (Draft): Investigation Volume and Planning Volume*, Sankaido Publishing.

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5. 3. 2 Design Method for Construction Spacing and Installation Angle of Drainage Pipes

Drainage pipes constructed from within the large-diameter drainage well are positioned to efficiently collect groundwater from the groundwater zone near the slip surface.



(a) Arrangement of drainage pipes in a slow-moving landslide cross-section

(b) Example of the plan layout of drainage

Figure 5.3.1 Example of large-diameter drainage well arrangement and drainage pipe arrangement

Figure 5.3.1(a) shows an example with a single-stage drainage pipes configuration. However, other groundwater zones within the mobile layer may also indirectly contribute to groundwater supply to the slip surface. To remove this groundwater, multiple drainage pipes may be arranged in stages from within the large-diameter drainage well.

The length of the drainage pipe is often around 50 m, with an additional 5 to 10 m of over-excavation beyond the target slip surface. Depending on the distance from the large-diameter drainage well to the target slip surface location and the construction capability of the boring equipment, the length may sometimes be set to 80 to 100 m. As shown in Figure 5.3.1(b), drainage pipes are arranged radially around the large-diameter drainage well, with a recommended tip spacing of 5 to 10 m. Japanese Specification B uses the hydraulic formula for pipe culverts to verify the appropriateness of the drainage pipe tip spacing.

References

PWRI (2008): T*echnical Guidelines for Slow-moving Landslide Prevention and Commentary*, Public Works Research Institute (PWRI), 142p.

JFCA (2013): *Technical Standards and Commentary for Forest Conservation: Slow-moving Landslide Prevention Edition*, Japan Forest Conservation Association (JFCA), 204p.

5. 3. 3 Design Method for Discharge Pipe and Placement of Large-diameter drainage wells

The length of discharge pipe is often set to approximately 80 meters or less, considering factors such as the construction capacity of the boring machinery. Discharge pipes must be positioned to avoid intersecting with slip surfaces. However, when direct drainage to the ground surface is difficult, large-diameter drainage wells are installed at locations 70 to 80 meters away and connected via discharge pipe. A dedicated relay well may also be installed.

Figure 5.3.2 shows the relationship between the discharge pipe and the well shaft. The required inner diameter of the discharge pipe is determined by comparing the discharge capacity based on the pipe diameter and slope i ($i = a/b$) with the discharge capacity based on the cross-sectional area S of the discharge opening inside the large-diameter drainage well and the water level height h from the center of that orifice; the smaller of the two discharge capacities is used to determine the pipe diameter. When there is no rainfall, the water level in the bottom tank forms near the inlet of the discharge pipe; however, the water level in the well rises due to rainfall or snowmelt. During torrential rain or massive snowmelt, the water level inside a large-diameter drainage well may rise higher than that in the storage tank (bottom tank); this indicates that the accumulated groundwater is not being discharged sufficiently, meaning the discharge pipe's capacity is insufficient. In such cases, measures such as installing additional discharge pipes are necessary. Specific design methods are described in Section 4.5.

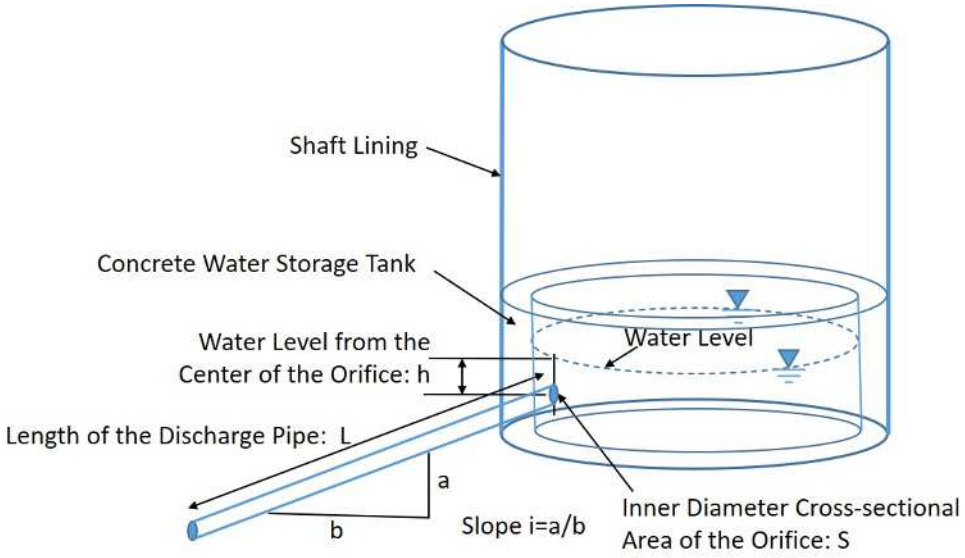


Figure 5.3.2 Factors Determining the Discharge Capacity of Discharge Pipe

5. 4 Examples of Items Requiring Attention When Developing Technical Standards

5. 4. 1 Diameter and Depth of Large-diameter Drainage Wells

1. The diameter of large-diameter drainage wells shall be determined by considering construction safety and the workability of drainage/ discharge boring operations.
2. The depth of large-diameter drainage wells shall extend to the impermeable layer below the slip surface. However, this does not apply to large-diameter drainage wells planned within areas of active slow-moving landslides.

【Explanation】

1) The inner diameter of the large-diameter drainage well is typically 3.5 m or 4.0 m, planned considering construction safety, depth, and the extension of the drainage/discharge pipe.

2) To remove groundwater near the slip surface and to position the discharge pipe within the impermeable layer below the slip surface, the bottom of the large-diameter drainage well is penetrated approximately 2 to 3 m into the impermeable layer. However, the bottom may be placed above the slip surface when: - Groundwater removal can be achieved without placing the bottom within bedrock; or - To prevent damage to the large-diameter drainage well due to slow-moving landslide activity.

Steel large-diameter drainage wells allow for some deformation due to slow-moving landslide activity. However, when using reinforced concrete (RC) large-diameter drainage wells that do not allow deformation, the potential for deformation or damage due to slow-moving landslide activity must be thoroughly evaluated.

【Reference Notes】

1) Relationship between the depth of the large-diameter drainage well and the depth of the slip surface

By placing the bottom below the slip surface, the discharge pipe can be positioned entirely within the stable layer. Placing the bottom within the mobile layer positions the discharge pipe within the mobile layer; however, since the mobile layer deforms with slow-moving landslide activity, the discharge pipe may be damaged. Near the end, collapse may occur within the mobile layer. Furthermore, by extending the shaft lining below the slip surface, it can penetrate not only the groundwater zones above and below the slip surface but also all groundwater zones within the mobile layer. This allows the pressure head of each groundwater zone at that location to be reduced to nearly zero, enabling the well to fully function as a fully penetrating well for collection. For steel large-diameter drainage wells, even if the shaft lining is destroyed by lateral shear at the slip surface depth due to slow-moving landslide activity, the drainage and discharge functions remain largely unaffected. Focusing on maximizing effectiveness as a slow-moving landslide countermeasure, the method of placing the bottom below the slip surface is superior. However, placing the well shaft below the slip surface and using vertical stiffeners simultaneously may potentially exacerbate deformation and damage to the shaft lining due to slow-moving landslide movement. Similarly, for reinforced concrete large-diameter drainage wells, placing the shaft below the slip surface increases the likelihood of deformation and damage associated with slow-moving landslide movement.

When slow-moving landslide is active and there is a possibility that the shaft lining could be severely damaged by landslide activity, the bottom should be set in the moving layer. However, in such cases, care must be taken in the placement to prevent damage to the discharge pipe due to deformation of the moving layer or the occurrence of cracks within it. Furthermore, designing discharge pipe to penetrate the slip surface during active slow-moving landslides should be avoided, as this would supply collected groundwater to the slip surface. While deformation

or damage to the shaft lining is easily detectable, confirming damage to the discharge pipe is difficult; minor bending fractures or crack leakage may go undetected.

5. 4. 2 Structure and Structural Calculations

The structure of the large-diameter drainage well shall be determined based on its purpose and site conditions, taking into account stability, constructability, and other factors.

【Explanation】

1. The structure of the large-diameter drainage well shall be designed for safety, considering ground conditions, constructability, and other factors.

   When using liner plates, reinforcement rings shall be used in conjunction as a general rule. Vertical stiffeners shall not be used, as they risk amplifying partial deformation of the large-diameter drainage well caused by slow-moving landslide activity into overall well deformation. Lateral struts shall also not be used, as they serve the same function as reinforcement rings and can be addressed through the placement and specifications of reinforcement rings. Another reason for not using vertical stiffeners or lateral struts is the absence of established design (structural calculation) methods for them.
2. The large-diameter drainage well shall be structurally designed to securely anchor its top and withstand settlement due to slope or its own weight.
3. The foundation's bottom tank (total height approx. 2m, depth approx. 1.5m) shall be constructed using watertight concrete or similar material to prevent re-infiltration of collected groundwater. Additionally, voids around the perimeter near the bottom tank shall be filled with grout to ensure collected groundwater reliably flows into the bottom tank.
4. Install an internal access staircase or ladder for maintenance of the large-diameter drainage well.
5. To prevent unauthorized access, construct the well shaft to rise approximately 0.5 to 1 meter above ground level. Install a lockable cover and manage the area appropriately by installing fencing or similar barriers around the perimeter.

【Reference Notes】

1) Calculation of Earth Pressure Acting on large-diameter drainage wells

While several formulas considering the effects of cylindrical shapes have been proposed for earth pressure acting on cylindrical excavation holes like large-diameter drainage wells, in the case of large-diameter drainage wells, a void occurs during construction due to over-excavation, and there is no close contact between the wall surface (shaft lining) and the excavation face. Therefore, the earth pressure formula for planar walls (the earth pressure formula that does not consider the friction angle of the wall surface) is often adopted.

The load acting on the outer surface of the large-diameter drainage well is considered to be earth pressure only. Water pressure is not considered because drainage holes are provided in the shaft lining. The method for calculating the earth pressure acting on the large-diameter drainage well should be selected from the following methods based on suitability for the specific site.

The following two earth pressure formulas have extensive application in Japan's slow-moving landslide countermeasure projects:

i) Method using Rankine's earth pressure formula (including multi-layer structures)

ii) Method using empirical earth pressure formulas

In Japan, the Terzaghi method, which accounts for the arch action of the surrounding soil, is sometimes used as an alternative. However, due to the lack of theoretical justification for its formula, it is preferable not to adopt it.

a) Rankine Earth Pressure Method (including multi-layer structures)

The Rankine earth pressure formula is as follows:

$$p=(\gamma \cdot h+q)\cdot \tan^2\left(45^\circ-\frac{\varphi}{2}\right)-2c\cdot tan\left(45^\circ-\frac{\varphi}{2}\right)$$

Here, p : Principal earth pressure at any depth h (m) (kN/m^2)

φ : Soil friction angle ($^\circ$)

c : Soil cohesion (kN/m^2)

γ: Soil unit weight (kN/m^3)

q: Superimposed load (kN/m^2)

For multi-layer structures, the active earth pressure on the upper surface of each layer, p_{iu}, and the active earth pressure on the lower surface, p_{id}, can be expressed by the following equations.

$$p_{iu}=\left(\sum_{j=1}^{j=i-1}\gamma_j\left(h_j-h_{j-1}\right)+q\right)\cdot\tan^2\left(45^\circ-\frac{\phi_i}{2}\right)-2c_i\cdot\tan\left(45^\circ-\frac{\phi_i}{2}\right)$$

$$p_{id}=\left(\sum_{j=1}^{j=i-1}\gamma_j\left(h_j-h_{j-1}\right)+\gamma_i\left(h_i-h_{i-1}\right)+q\right)\cdot\tan^2\left(45^\circ-\frac{\phi_i}{2}\right)-2c_i\cdot\tan\left(45^\circ-\frac{\phi_i}{2}\right)$$

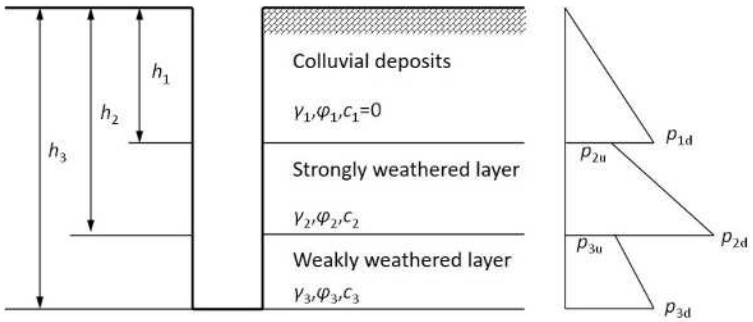


Figure 5.4.1 Rankine's Earth Pressure Distribution (Multi-Layered Soil Structure)

b) Method Using Empirical Earth Pressure Formula

In Japan's empirical earth pressure calculations for large-diameter drainage well works, a method is frequently used to determine the static earth pressure p_h, assuming that earth pressure does not increase beyond depths of approximately 15m (see Figure 5.4.2). This earth pressure formula yields a constant earth pressure distribution independent of geological conditions, but employs an earth pressure coefficient based on the worst-case geological condition of clayey soil. In deep sections of high-quality weathered rock formations, soil pressure may become negligible. However, applying this formula to such formations would result in an overestimation of soil pressure.

Therefore, this formula should be applied only when estimating the c and φ values of the target soil is difficult, or when soil conditions are poor, such as in clayey soils or colluvial deposits. This formula cannot be used when thick layers of high-quality bedrock are present.

$p_h = k \cdot (\gamma \cdot h + q)$ $(h < 15\text{m})$

$p_h = p_0$ $(h \geqq 15\text{m})$

Here, p_h: Earth pressure (kN/m²)

k: Static earth pressure coefficient (0.5)

h: Depth from ground surface (m)

q: Overburden load (kN/m²)

p_0: Earth pressure at h = 15 m (kN/m²)

Furthermore, if it is determined that earth pressure will increase even at depths greater than h = 15 m, the depth at which earth pressure is assumed not to increase may be set deeper than 15 m.

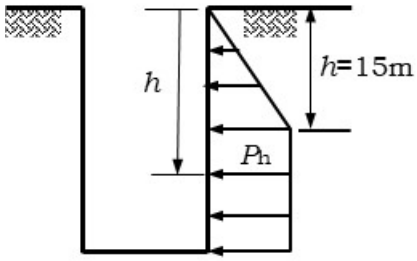


Figure 5.4.2 Empirical Earth Pressure Distribution Frequently Used in Japanese Large-diameter drainage well Design

d) Analysis of Partial Earth Pressure

When using a steel large-diameter drainage well and setting the elliptical deformation amount δ_0 of the shaft lining to 4–10 cm, considering the excess excavation during large-diameter drainage well construction, partial earth pressure does not need to be considered. When adopting an RC drainage well, the partial earth pressure shall be modeled as a triangular distribution—ranging from zero at the ground surface to approximately 100 kN/m² at a depth of 15 m—and as a constant value at depths of 15 m and greater.

2) Structural Calculation for Steel Structure (Liner Plate) Large-diameter drainage well

<Design Conditions>

i) The structure shall combine the liner plate and reinforcement ring.

ii) The reduction rate in the cross-sectional performance of the liner plate due to drainage holes, etc., shall be as follows.

(Cross-sectional area A: 20%, Section modulus Z: 20%, Second moment of area I: 20%)

iii) The shaft lining elliptical deformation amount δ_0 shall be 4 to 10 cm, considering the over-excavation allowance during the large-diameter drainage well construction.

a) Distribution Ratio of Section Forces (Axial Force, Bending Moment)

The distribution ratio of section forces between the liner plate and the reinforcement ring shall be calculated using the following formula.

$$\alpha_L = \frac{A_L}{\left(A_L + \frac{A_H}{l}\right)} \quad , \quad \alpha_H = \frac{A_H / l}{\left(A_L + \frac{A_H}{l}\right)}$$

$$\beta_L = \frac{I_L}{\left(I_L + \frac{I_H}{l}\right)} \quad , \quad \beta_H = \frac{I_H / l}{\left(I_L + \frac{I_H}{l}\right)}$$

Here, α_L: Axial force distribution ratio of the liner plate

α_H: Axial force distribution ratio of the reinforcement ring

β_L: Bending moment distribution ratio of the liner plate

β_H: Bending moment distribution ratio of the reinforcement ring

A_L: Cross-sectional area of the liner plate per 1m depth (m^2/m)

A_H: Cross-sectional area per reinforcement ring (m^2)

I_L: Second moment of area per meter of liner plate (m^4/m)

I_H: Second moment of area per meter of reinforcement ring (m^4/m)

l: Reinforcement ring spacing (m)

b) Buckling Analysis

<Allowable Buckling Load>

The allowable buckling load q_a when the ring is subjected to uniformly distributed load is calculated using the following formula.

$$q_a = \frac{3 \cdot E \cdot I}{f \cdot r^3} \geqq p_{max}$$

Here, q_a: Allowable buckling load (kN/m^2)

E: Young's modulus ($=2.0\times10^8$) (kN/m^2)

I: Second moment of area per 1m depth of outer frame material (m^4/m)

$$I = I_L + \frac{I_H}{l}$$

I_L: Second moment of area per 1 m depth of the liner plate (m^4/m)

I_H: Second moment of area of the reinforcement ring (m^4/m)

l: Reinforcement ring spacing (m)

f: Safety factor (1.5)

r: Well radius (m)

p_{max}: Maximum earth pressure acting on the outer surface of the sump well (kN/m^2)

＜Critical Buckling Load＞

$$q_{cr}=\frac{3\cdot E\cdot I}{r^3}$$

Here, q_{cr}: Critical buckling load (kN/m²)

c) Analysis of Cross-Section Forces

<Axial Force>

$$N=p_{max}\cdot r$$

Here, N: Axial force (kN/m)

p_{max}: Design earth pressure (kN/m²)

r: Radius of the large-diameter drainage well (m)

＜Maximum Bending Moment＞

$$M_{max}=N\cdot\frac{\delta_0}{1-\frac{p_{max}}{q_{cr}}}$$

Here, M_{max}: Maximum bending moment (kN·m/m)

N: Axial force (kN/m)

δ_0: Elliptical deformation (m) (0.04 to 0.1 m)

p_{max}: Design earth pressure (kN/m²)

q_{cr}: Limit buckling load (kN/m²)

Due to the presence of excavation surplus voids around the perimeter of the large-diameter drainage well, significant earth pressure can cause elliptical deformation of the outer ring. Therefore, the elliptical deformation amount δ_0 must account for the excavation surplus during construction. Set the elliptical deformation amount considering the range including the wave height of the liner plate and the bolts for fixing the reinforcement ring (a+b+c in Figure 5.4.3). Generally, 5 cm is often used.

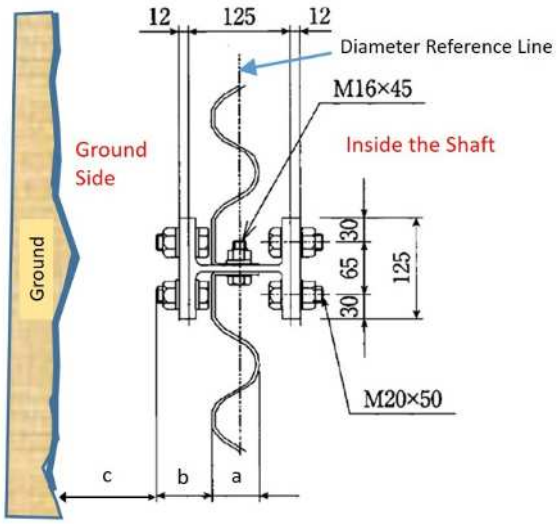


Figure 5.4.3 Structural Steel Related to Overditching of the Large-diameter drainage well

In Japan, a method using 1% of the radius *r* of the large-diameter drainage well as the elliptical deformation is sometimes adopted, but this is not documented in Japan's official technical standards. This figure, used in preliminary structural calculations around 1980, is still in use today, and this "1% of radius *r*" has no theoretical basis. Furthermore, for a large-diameter drainage well with a radius of 1.75 m, the elliptical deformation becomes a small value of 1.75 cm. Consequently, the maximum bending moment calculated using the above formula also becomes small, allowing the use of thinner steel plates and resulting in a design that leans toward the unsafe side.

d) Stress Verification

The bending stress verification for the liner plate and reinforcement ring shall be performed using the following formula.

$$\sigma_L = \frac{\alpha_L \cdot N}{A_L} + \frac{\beta_L \cdot M_{\max}}{Z_L} \leqq \sigma_{La}$$

$$\sigma_H = \left(\frac{\alpha_H \cdot N}{A_H} + \frac{\beta_H \cdot M_{\max}}{Z_H} \right) \cdot l \leqq \sigma_{Ha}$$

Here, σ_L: Bending stress in the liner plate (N/mm²)

σ_H: Bending stress in the reinforcement ring (N/mm²)

σ_{La}: Allowable bending stress in the liner plate (N/mm²)

σ_{Ha}: Allowable bending stress in the reinforcement ring (N/mm²)

A_L: Cross-sectional area of the liner plate per meter of depth (mm²/m)

A_H: Cross-sectional area per reinforcement ring (mm²)

Z_L: Section modulus of the liner plate per meter of depth (mm³/m)

Z_H: Section modulus per reinforcement ring (mm³)

α_L: Axial force contribution ratio of the liner plate

α_H: Axial force contribution ratio of the reinforcement ring

β_L: Bending moment contribution ratio of the liner plate

β_H: Bending moment contribution ratio of the reinforcement ring

l: Spacing between reinforcement rings (m)

3) Design of RC Large-diameter drainage wells

This large-diameter drainage well is generally used in relatively homogeneous geological formations where durability is required due to corrosive environments, in ground that cannot support itself during excavation, in sandy formations prone to boiling due to high groundwater levels, and in soft clay formations prone to heaving. Here, we introduce a design method based on HCA (2010) design manual with slight modifications. This design method centers on partial earth pressure, making the determination of partial earth pressure essential.

a) Distribution of Static Earth Pressure and Partial Earth Pressure

Design assumes partial earth pressure acting in one direction in addition to static earth pressure acting on the shaft lining. Japanese standards for caisson foundations in road bridges assume half the static earth pressure acts.

Note that significant superimposed loads requiring consideration typically do not occur in slow-moving landslide areas.

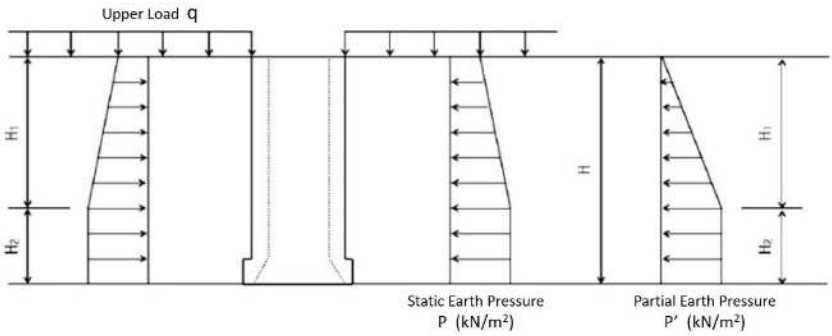


Figure 5.4.4 Design Earth Pressure Diagram (HCA, 2010)

b) Calculation of Earth Pressure and Internal Forces Acting on Circular Beams

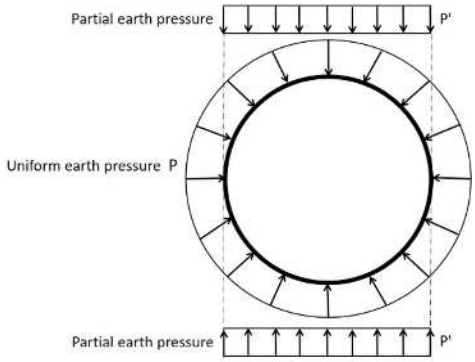


Figure 5.4.5 Partial Earth Pressure and Uniform Earth Pressure Acting on Shaft Lining (Enokida, 2026)

The bending moment M_φ and axial force N_φ per shaft lining block, when uniform earth pressure P and partial earth pressure P' act, are expressed as functions of angle φ as shown in Figure 5.4.6 by the following equations.

$$M_\phi = \frac{P' \cdot r^2}{2}\left(\sin^2\phi - \frac{1}{2}\right) \cdot B$$

$$N_\phi = P' \cdot r \cdot \cos^2\phi \cdot B + P \cdot r \cdot B$$

Here, M_φ: Bending moment (kN·m), N_φ: Axial force (kN), φ: Angle of the internal angle at the target position (°), P: Uniform earth pressure (kN/m²), P': Partial earth pressure (kN/m²), r: Shaft lining radius (m), B: Shaft lining block height (m)

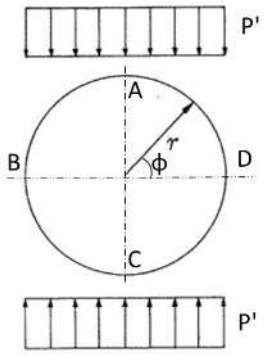


Figure 5.4.6 Distribution of Partial Earth Pressure and Cross-Section Force Verification Positions for the Shaft lining

Assuming the ring is made of homogeneous material, determine the bending stress at each point (A, B, C) acting on the rectangular cross-section (vertical cross-section) of the ring. Designate the point where the bending stress is maximum as the verification point, and define the bending moment and axial force at that point as the maximum bending moment M_{max} and maximum axial force N_{max}.

$\sigma_{A0} = N_{90}/A + M_{90}/Z$

$\sigma_{B0} = N_{180}/A + M_{180}/Z$

$\sigma_{C0} = N_{270}/A + M_{270}/Z$

Here, σ_{A0}, σ_{B0}, σ_{C0}: Bending stress at each point of the ring (kN/m²)

A: Cross-sectional area of the ring's rectangular cross-section (m²) ($=H \times B$)

Z: Section modulus of the ring's rectangular cross-section (m³) ($=B \cdot H^2/6$)

H, B: Cross-sectional dimensions (m) of the unit segment of the ring (cross-sectional dimensions of the unit segment of the RC segment)

c) Circular Cross-Section Analysis

Analysis shall be performed for a rectangular cross-section with multiple reinforcing bars.

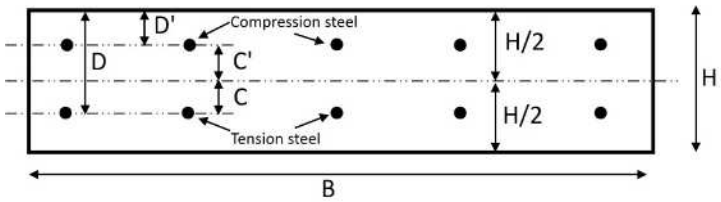


Figure 5.4.7 Cross-section of a ring (Enokida, 2026)

Distance e (m) from the geometric center of the cross-section to the center of gravity of the axial force

$e = M_{max}/N_{max}$

Distance C (m) from the center of the member thickness to the tensile steel

$C = D - H/2$

Here, D: Effective height of tension steel (m), H: Member thickness (m)

Distance C' (m) from the center of the member thickness to the compression reinforcement

$C' = H/2 - D'$

Here, D': Effective height of compression steel (m)

In a standard large-diameter drainage well, the center of gravity position e of the axial force becomes greater than $H/2$, and since the center of gravity position of the axial force is judged to be off-center, the design calculation formula for a double-reinforced rectangular cross-section subjected to an off-center load N is adopted.

Using the steel area (tension side A_s, compression side A_s') (m²) and the reinforcement ratio n (=15), the distance X to the neutral axis of the rectangular section subjected to eccentric loading N (kN) off-center is determined from the following equation.

$$X^3 - 3\left(\frac{H}{2} - e\right)X^2 + \frac{6n}{B}[A_s(e + C) + A_s'(e - C')]X - \frac{6n}{B}[A_s D(e + C) + A_s' D'(e - C')] = 0$$

Transformed second moment of area

$$I_{cr} = \frac{BX^2}{2}\left(\frac{H}{2} - \frac{X}{3}\right) + nA_s'C'(X - D') + nA_sC(D - X)$$

Verification of Concrete Compressive Stress Level

$$\sigma_c = \frac{M}{I_{cr}}X \leqq \sigma_{ca}$$

Verification of Tensile Stress Level for Tension Steel

$$\sigma_s = n\sigma_c\frac{(D - X)}{X} \leqq \sigma_{sa}$$

Verification of Tensile Stress Level for Compression Steel

$$\sigma_s' = n\sigma_c\frac{(X - D')}{X} \leqq \sigma_{sa}$$

d) Longitudinal Joint Considerations

The design of reinforcing bars or bolts connecting rings longitudinally shall account for shear forces arising from partial earth pressure. Assuming partial earth pressure acts over the entire depth would result in enormous shear forces; therefore, it is assumed that partial earth pressure acts within the range of the shaft lining block height B. The shear force S (kN) acting on the horizontal cross-section (ring cross-section) of the shaft lining is calculated using the following formula.

$S = 2 \times r \times B \times P'$

Here, r: Outer diameter of the shaft lining block (m)

B: Height of the shaft lining block (m)

P': Partial earth pressure (kN/m²)

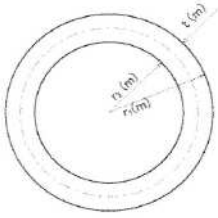


Figure 5.4.8 Horizontal Cross-Section of the Ring

Calculate the shear stress τ (kPa) per bar of reinforcing steel or bolt using the following formula, and design so that the allowable shear stress τ_a (kPa) is not exceeded.

$$\tau = \frac{S}{A_s \cdot i} \leqq \tau_a$$

Here, A_s: Nominal cross-sectional area per bar of rebar or bolt (m²)

i: Number of bars of rebar or bolts

e) Examination of Segment Joints (Circumferential Joints)

While the design manual of HCA (2010) employs design equations for steel segments, RC segments combine concrete and reinforcing steel—materials with differing elastic properties—thus requiring different design equations than those for steel segment joints. Following JSCE (2010), joints for RC segments are designed using conventional calculation methods. First, the resistance moment of the main cross-section shown in Figure 5.4.9 is calculated using the following formula.

$$x = -\frac{n(A_s + A'_s)}{B} + \sqrt{\left\{\frac{n(A_s + A'_s)}{B}\right\}^2 + \frac{2n}{B}(A_s d + A'_s d')}$$

$$M_{rc} = \left\{\frac{Bx}{2}\left(d - \frac{x}{3}\right) + nA'_s x \frac{x - d'}{x}(d - d')\right\}\sigma_{ca}$$

$$M_{rs} = \frac{\left\{\frac{Bx}{2}\left(d - \frac{x}{3}\right) + nA'_s x \frac{x - d'}{x}(d - d')\right\}x}{n(d - x)}\sigma_{sa}$$

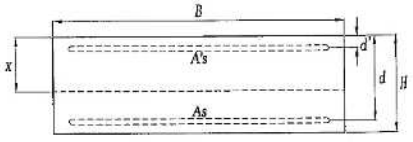


Figure 5.4.9 Main Cross-Section of the Ring

Here, M_{rc}: Resistance moment at which concrete reaches allowable stress (kN·m)

M_{rs}: Resistance moment at which tensile steel reaches allowable stress (kN·m)

B: Height of shaft lining block (m)

H: Thickness of shaft lining block (m)

n: Young's modulus ratio (n=15)

E_s: Young's modulus of steel (kN/m²)

E_c: Young's modulus of concrete (kN/m²)

A_s: Cross-sectional area of tensile steel (m²)

A'_s: Cross-sectional area of compressive steel (m²)

d: Effective height of tensile steel (m)

d': Effective height of compressive steel (m)

x: Distance from the compressive side edge of the shaft lining block to the center axis (m)

σ_{ca}: Allowable stress of concrete (kN/m²)

σ_{sa}: Allowable stress of steel (kN/m²)

The moment of resistance M_r for the main section shall be taken as the smaller value of M_{rc} or M_{rs}.

Next, the moment of resistance for the joint section shown in Figure 5.4.10 shall be calculated using the following formula.

$$x = -\frac{nA_B}{B}\left(-1 + \sqrt{1 + \frac{2Bd}{nA_B}}\right)$$

$$M_{jrc} = \frac{1}{2} \cdot Bx\left(d - \frac{x}{3}\right) \cdot \sigma_{ca}$$

$$M_{jrb} = A_B \cdot \left(d - \frac{x}{3}\right) \cdot \sigma_{Ba}$$

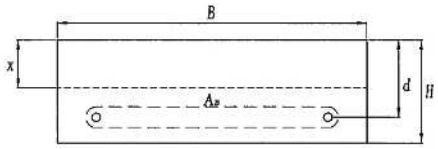


Figure 5.4.10 Joint Cross-Section

Here, M_{jrc}: Resistance moment at which concrete reaches allowable stress (kN·m)

M_{jrb}: Moment of resistance at which the bolt reaches its allowable stress (kN·m)

A_B: Cross-sectional area of the bolt (m²)

d: Effective height of the bolt (m)

x: Distance from the compressed end of the shaft lining block to the center axis (m)

σ_{ca}: Allowable stress of concrete (kN/m²)

σ_{Ba}: Allowable stress of the bolt (kN/m²)

The moment of resistance M_{jr} of the joint cross-section shall be the smaller of M_{jrc} or M_{jrs}.

Since the moment of resistance M_{jr} of the joint cross-section must be at least 60% of the moment of resistance M_r of the main cross-section, it must satisfy the following equation.

$$0.6M_r \leqq M_{jr}$$

References

TACPL (2009): *Liner Plate Design and Construction Manual*, Technical Association of Corrugated Pipes and Liner Plates (TACPL)

HCA (210): *Reinforced Concrete Large-diameter Design and Construction Manual*, Hokuriku Construction Association (HCA), 71p.

JSCE (2010): *Design of Segment for Shield Tunnel (Revised version)*, Tunnel Library 23, Japan Society of Civil Engineers (JSCE).

Nakamura, H., Shiroishi, Y. (1980): Structural Design of Large-diameter Drainage Wells in Slow-moving Landslide Areas, Public Works Research Institute (PWRI), *Technical Note of PWRI*, No.1618.

Enokida, M. (2026): Proposal for improving structural calculation methods for RC drainage wells in Japan, *J. of the Jpn. Landslide Soc.*, Vol.63, No.1, pp.11-16.

5. 5 Examples of Other Technical Standards

5. 5. 1 Calculation of Groundwater Level Lowering Effect

The calculation of the effect of large-diameter drainage wells shall be performed using an appropriate groundwater analysis method that considers their layout and number.

【Explanation】

The effectiveness of large-diameter drainage wells should be evaluated based on the reduction in pore water pressure acting on the slip surface. However, since it is difficult to determine this directly, the effectiveness is instead evaluated based on the reduction in groundwater level within the aquifer directly affecting the pore water pressure on the slip surface.

The large-diameter drainage well method primarily involves collecting groundwater within the wellbore via drainage pipes and discharging it through discharge pipe. However, since the well shaft with drainage holes also penetrates multiple groundwater zones present at its installation location, a certain degree of collection effect can be expected, leading to an anticipated reduction in the groundwater level around the large-diameter drainage well.

The range of the groundwater level lowering effect of the large-diameter drainage well structure varies depending on the arrangement and number of large-diameter drainage wells and changes with distance from the wells, meaning the lowering is not uniform. The groundwater level lowering effect is calculated using groundwater analysis methods that consider the location and number of large-diameter drainage wells, as well as the mutual effects between multiple large-diameter drainage wells. Furthermore, the groundwater level lowering effect varies depending on whether the large-diameter drainage well is a fully penetrating well or a partially penetrating well, so this must be considered in the calculation.

When examining the increase in the safety factor due to the groundwater level lowering effect using a two-dimensional stability analysis of only the main section, it is necessary to perform the stability analysis considering the average groundwater level lowering in the transverse direction to appropriately evaluate the effect of the large-diameter drainage well at locations distant from the main section.

【Reference Notes】

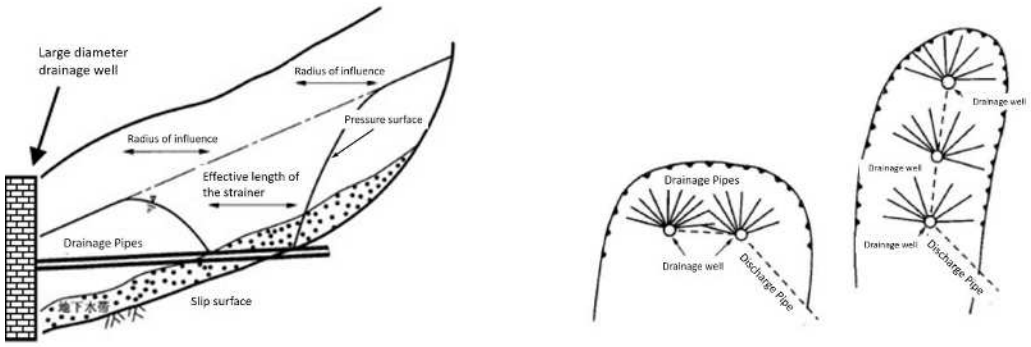
Accurately calculating the groundwater level reduction effect on the target aquifer caused by the large-diameter drainage well structure and drainage pipes is difficult. In Japan's Specification B, a conventional method is adopted. This involves calculating the effect of the large-diameter drainage well itself using groundwater well formulas to estimate the planar distribution of groundwater level decline. Based on the calculated large-diameter drainage well's own collection volume Q and the rainfall intensity during rainfall, the number of drainage pipes and other parameters are determined using culvert formulas to enable the collection of the directly supplied groundwater volume. This conventional practice was developed under the assumption that, even when combining the collection functions of both the well shaft and the drainage pipes, the collection capacity in slow-moving landslide areas is only equivalent to or less than that of a well shaft in a flat groundwater basin. For more detailed analysis, three-dimensional seepage flow analysis using methods such as the finite element method is also effective.

5. 5. 2 Design of Drainage Boring Works

The location, number, direction, and length of the drainage pipes shall be determined so as to effectively collect groundwater from points where it concentrates on the slip surface or from the aquifer.

【Explanation】

The drainage pipes constructed from within the large-diameter drainage well are positioned to efficiently collect groundwater from the groundwater zone near the slip surface.



(a) Arrangement of drainage pipes in a slow-moving landslide cross-section

(b) Example of the plan layout of drainage pipes

Figure 5.5.1 Example of drainage pipes arrangement

Figure 5.5.1(a) shows an example with a single-stage drainage pipes arrangement. However, other groundwater zones within the mobile layer may also indirectly contribute to groundwater supply to the slip surface. To exclude this groundwater, multiple stages of drainage pipes may be arranged within the large-diameter drainage well.

The length of drainage pipes is typically around 50 m, with an additional 5 to 10 m of drilling beyond the target slip surface. Depending on the distance from the large-diameter drainage well to the target slip surface location and the construction capability of the boring equipment, the length may sometimes be set at 80 to 100 m. As shown in Figure 5.5.1(b), drainage pipes are arranged radially around the large-diameter drainage well, with a recommended tip spacing of 5 to 10 m. Japanese Specification B uses the hydraulic formula for pipe culverts to verify the adequacy of the drainage pipes tip spacing.

【Reference Notes】

One method for estimating the effectiveness of drainage pipes and theoretically determining their spacing involves utilizing hydraulic formulas for pipe culverts. While accurately capturing complex groundwater behavior is difficult, this approach is effective for roughly determining the spacing of drainage pipes and their hydraulic head reduction effect. Another method involves using finite element method seepage analysis.

1. Hydraulic Formula for Culverts

(1) For Pressurized Groundwater

The borehole spacing $2d$ within the effective strainer section is expressed by the following equation (Koyanagi-Maekawa formula) based on the relationship with the water level drop height s at the midpoint of the borehole.

$$d = \frac{2b_w}{\pi} \sinh^{-1}[\exp(x)]$$

$$\theta = 2\sin^{-1}\frac{d}{a}$$

$$x = \ln\left[\sinh\left(\frac{\pi r_0}{2b_w}\right)\right] + \pi k\frac{s_0 - s/2}{q}$$

$$q = \pi k\frac{s_0}{\ln\dfrac{\sinh\left(\frac{\pi R}{2b_w}\right)}{\sinh\left(\frac{\pi r_0}{2b_w}\right)}}, \qquad Q = qL_s$$

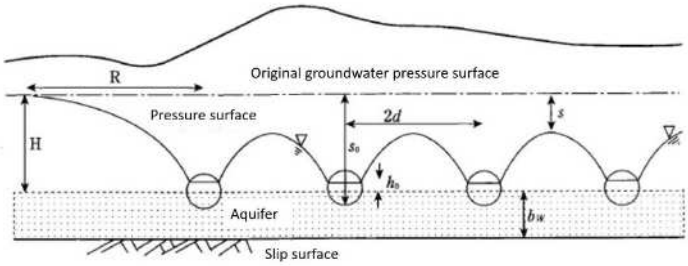


Figure 5.5.2 Schematic of Groundwater Pressure Distribution Around a Culvert

$$R = 575 \cdot s_0\sqrt{kb_w}$$

Here, q: Collection volume per unit strainer length (m³/s/m)

Q: Total collection volume (m³/s), L_s: Effective strainer length (m)

s_0: Water level drop height at culvert location = H - h_0 (m) ($h_0 \approx r_0$)

r_0: Pipe culvert radius (m), b_w: Groundwater zone thickness (m)

k: Permeability coefficient (m/s), R: Influence radius (m)

s: Water level drop height at the midpoint of the borehole (m)

a: Distance from the large-diameter drainage well to the midpoint of the strainer section (m)

d: Half the spacing between drainage pipes tips (m)

(2) For Free Groundwater

By deriving the A.N. Koschakov equation, the relationship between the pipe culvert tip spacing $2d$ and the average water level height h_{bar} is obtained by the following equation.

$$h_{bar} = \frac{q_0 + wd}{\alpha k}\left(\ln\frac{d}{r_0} + \frac{r_0}{d} - 1\right)$$

$$\alpha = \frac{\pi}{2} + h_{max}/d$$

$$h_{max} = \frac{1}{2}\sqrt{\left(\frac{\pi d}{2}\right)^2 + \frac{(q_0 + wd)\cdot 4d}{k}\cdot ln\frac{d}{r_0}} - \frac{\pi d}{4}$$

$$q_0 \fallingdotseq \alpha_0 k\frac{H_1}{ln\frac{R}{r_0}}, \quad \alpha_0 \fallingdotseq \frac{\pi}{2} + \frac{H_1}{R}$$

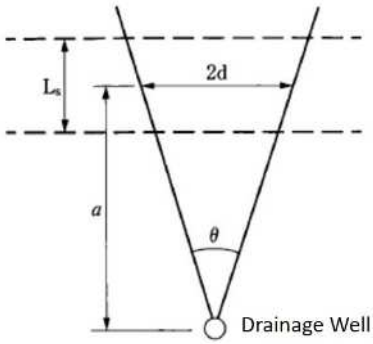


Figure 5.5.3 Spacing of Drainage Pipes Tips

$$R = 575 \cdot s_0\sqrt{kH}$$

Here, h_{bar}: Average water level measured from the culvert elevation (m)

h_{max}: Maximum water level measured between culverts from the culvert elevation (m)

q_0: Inflow per unit length when rainfall infiltration is not considered (m³/s/m)

w: Infiltration capacity (rainfall intensity equivalent) (m/s)

k: Permeability coefficient (m/s), R: Influence radius (m)

H_1: Original water level height relative to the culvert elevation (m)

5. 5. 3 Design of Discharge Pipe

Discharge pipe shall, in principle, be installed in the direction of the shortest possible length with a gradient allowing natural flow.

【Explanation】

The length of discharge pipe is often set to approximately 80 meters or less, considering factors such as the construction capacity of the drilling machinery. Discharge pipes must be positioned to avoid intersecting with slip surfaces. However, when direct discharge to the ground surface is difficult, a large-diameter drainage well is installed at a distance of 70 to 80 meters and connected to the discharge pipe. In some cases, a dedicated relay well is also installed.

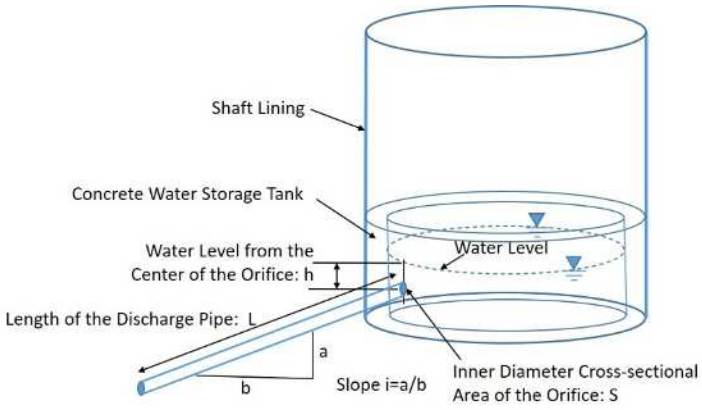


Figure 5.5.4 Factors Determining the Discharge Capacity of Discharge Pipe

Figure 5.5.4 illustrates the relationship between the discharge pipe and the well shaft. The required inner diameter of the discharge pipe is determined by comparing two discharge capacities: one based on the pipe's diameter and slope i ($i = a/b$), and the other based on the cross-sectional area S of the discharge outlet within the large-diameter drainage well and the water level height h from the center of the inlet. The diameter is determined by the smaller value of these two discharge capacities. During torrential rain or massive snowmelt, the groundwater level inside the large-diameter drainage well may rise above the level of the stilling basin. This indicates that the accumulated groundwater is not being adequately discharged, which is due to insufficient discharge capacity of the discharge pipe. In such cases, measures such as installing an additional discharge pipe are necessary.

【Reference Notes】

1. Hydraulic Formulas Used in Design

The following hydraulic formulas are sometimes used to determine the appropriate diameter of a discharge pipe.

(1) Hazen-Williams equation (formula for calculating discharge capacity based on pipe diameter and slope)

$$Q = 0.84935 \cdot A \cdot C \cdot R^{0.63} \cdot I^{0.54}$$

Here, Q: Flow rate (m³/s), A: Pipe cross-sectional area (m²), C: Velocity coefficient (=100)

R: Depth (m) = $D/4$, D: Pipe diameter (m), I: Hydraulic gradient

(2) Manning's formula (formula for calculating discharge capacity based on the diameter and slope of the discharge pipe)

$$Q = \frac{1}{n} A \cdot R^{2/3} \cdot I^{1/2}$$

Here, n: Manning's roughness coefficient (=0.012)

(3) Relationship Between the Water Level in the Well and the Discharge Rate from the Discharge Pipe (Formula for Calculating Discharge Capacity Based on the Outlet Area and the Water Level in the Well Shaft)

The flow rate based on the discharge capacity corresponding to the well water level is calculated using the following equation, which is derived from the Darcy-Weisbach equation. The equation for estimating the friction loss coefficient λ of the discharge pipe, which accounts for turbulent flow, is shown below; this equation is commonly used in civil engineering applications such as open channels and sewer systems.

$$\mathrm{Q} = \frac{1}{\sqrt{1 + f_e + \lambda(L/D)}} A\sqrt{2gH}$$

$$\lambda = 124.5n^2/D^{1/3}$$

Here, g: gravitational acceleration (m/s²), H: height from wellbore water level to centerline of discharge pipe (m)

f_e: inflow loss coefficient (=0.5), λ: friction loss coefficient, L: pipe length (m)

2. Using Hydraulic Formulas to Determine Discharge Pipe Diameter

(1) Setting the Planned Discharge Volume and Calculating the Required Discharge Pipe Diameter

Calculate using the following steps:

1) Estimate the planned discharge volume based on the maximum rainfall for the site and the catchment area of the large-diameter drainage well. If the water level inside the well shaft is being monitored, the maximum inflow volume described later can be used as the planned discharge volume.
2) Assume the diameter, length, and slope of the discharge pipe.
3) Calculate each flow rate Q using the three hydraulic formulas above and adopt the smallest flow rate. Note that when using the hydraulic formula in (3) above, H is the height from the centerline of the discharge pipe to the top surface of the bottom tank.
4) Confirm that the selected flow rate exceeds the planned discharge volume. If the flow rate falls below the planned discharge volume, change the discharge pipe specifications and repeat steps 2) to 3) above.

(2) Calculation of Maximum Inflow

Observe the water level inside the well during torrential rainfall or massive snowmelt periods. Using the hydrological formula described in "(3) Relationship Between Water Level Inside the Well and Discharge Volume from the Discharge Pipe" above, calculate the flow rate Q based on the observed maximum water level h_{max} and the height H from the well water level to the center of the discharge pipe. This flow rate Q represents the maximum inflow to the large-diameter drainage well.

6. Concrete Crib Works Design

6. 1 Overview of Concrete Crib Works

6. 1. 1 Purpose of Concrete Crib Works

Concrete crib works are a representative method of slope protection using structures in Japan. They are installed on cut slopes, embankment slopes, or unstable slopes to prevent weathering and erosion of the slope and to prevent small-scale failures.

6. 1. 2 Effectiveness of Concrete crib works

Concrete crib works inherently prevent slope erosion and small-scale collapses. Additionally, covering the interior enhances protection against weathering and erosion of the underlying materials. Implementing vegetation within the frames helps maintain the environmental landscape. Furthermore, for failures too large to be restrained by concrete crib works alone, combining them with restraint measures such as slope reinforcement works or ground anchors enables handling failures of a certain scale. However, this document explains the design when concrete crib works are used independently.

Regarding slope protection works that enhance the prevention of weathering and erosion of the natural slope, sprayed concrete is one option. However, it is often structurally unreinforced mortar, making it prone to cracking due to drying shrinkage. It also has low durability, lacks small-scale collapse prevention functionality, and can disrupt the landscape due to full slope coverage. Therefore, concrete crib works offer superior workability, durability, and aesthetic appeal.

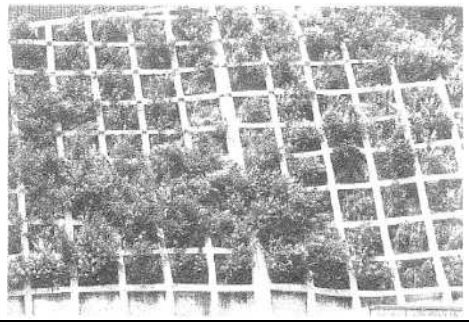
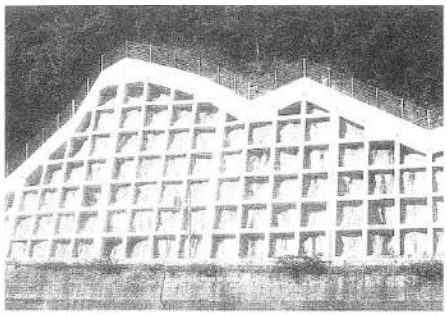
6. 1. 3 Structure of Concrete Crib Works

Concrete crib works can be primarily classified into shotcrete crib works and cast-in-place crib works. Table 6.1.1 shows the structural characteristics of each concrete crib works.

Shotcrete crib works are structures where wire mesh or similar formwork is installed in a grid pattern on the target slope, and then mortar or concrete is sprayed to construct the grid-like frame. Compared to cast-in-place concrete crib works, shotcrete uses lightweight formwork components, offers better workability, and can accommodate uneven slopes or steep slopes with large height. Furthermore, the widespread adoption of construction machinery for the “wet shotcrete method”—which produces less rebound and facilitates easier mix design control—has made shotcrete the most commonly used concrete crib works.

The cast-in-place concrete crib works involves installing wooden or other formwork and reinforcing bars in a grid pattern on the target slope, then placing and curing concrete to form the grid-shaped frame. While it offers advantages over the sprayed concrete method, such as not requiring specialized machinery, its workability—particularly in terms of adhesion to the natural ground, formwork installation, and reinforcing bar placement on high or uneven slopes—is inferior to the shotcrete crib works.

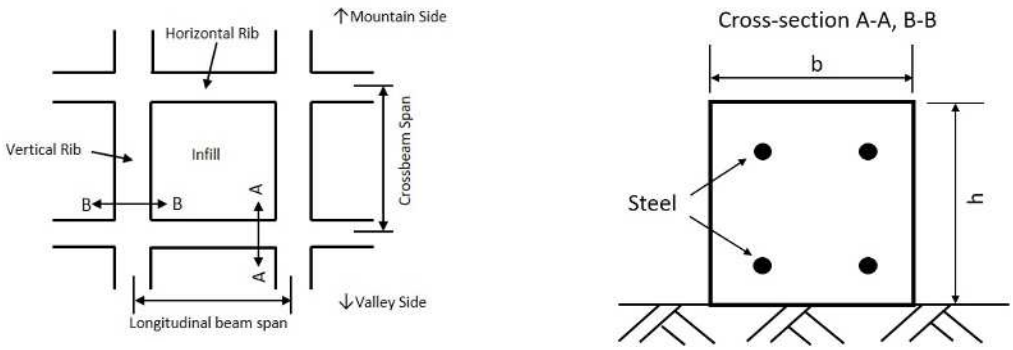
Table 6.1.1 Types and Characteristics of Concrete Crib Works (Modified from JSPA,2006, 2013)

	Shotcrete crib Works	Cast-in-Place Concrete Crib Works
Advantages	・The continuous intersections of the formwork provide high bending strength. ・Some degree of suppression effect against surface sliding can be expected. ・High adhesion between the formwork and the natural ground provides strong resistance to scouring. ・Construction is possible even on high or uneven slopes.	・The continuous intersections of the formwork provide high bending strength. ・Offers some degree of suppression against surface sliding. ・Can be constructed on slopes with fairly steep gradients.
Disadvantages	・Requires skilled labor as it is a manual construction method. ・Requires rebound treatment and final surface finishing.	・Formwork installation and reinforcement placement are difficult on high or uneven slopes, and uniform concrete placement is challenging. ・The bond between the formwork and the ground is inferior to that of shotcrete crib works. ・Small-section concrete crib works are difficult to construct. ・Requires foundation concrete. ・Formwork support is necessary, requiring step-by-step construction from the lower sections, making it a complex operation. ・Construction period is longer than shotcrete crib works.
Construction Examples		

Hereafter, we will focus on shotcrete crib works, which offer numerous advantages as slope protection works, and explain the frame size and spacing.

Similar to shotcrete works widely adopted overseas, shotcrete crib works possess erosion control functions. Furthermore, they utilize the space within the frames to function as a greening foundation work (preventing the loss of vegetative soil) (Figure 6.1.2).

Figure 6.1.1 shows the names of the main components of the shotcrete crib works. The standard shape and dimensions of the shotcrete crib works are set to the values shown in Table 6.1.2 as empirical guidelines for erosion control and as a greening foundation work. When performing soil replacement or sandbag works within the frame, a minimum frame cross-section of 150 mm × 150 mm is used to provide the thickness necessary for plant growth.



(a) Front View **(b) Rib Cross-Section**

Figure 6.1.1 Rib cross-section of a shotcrete crib works and names of main components (FFS, 1983)

Table 6.1.2: Guidelines for Shotcrete Crib Works Specifications as Erosion Control and Revegetation Foundation Works (Based on FFS, 2008)

Performance Requirement Category	Application Purpose	Frame Size (mm) (*b*×*h*)	Frame Spacing (m) (Vertical Rib × Horizontal Rib)	Remarks
Slope Control Works	Rainfall Erosion Control (When Sprayed Within Frame)	150×150	1.15×1.15 ～ 1.5×1.5	Use as standalone shotcrete crib work
	Use as revegetation foundation work	150×150 (200×200)		

Under the following conditions, a cross-section of 200mm × 200mm or larger is desirable:

- When the slope has severe irregularities or numerous cracks, and the effectiveness of preventing soil collapse is considered low.
- When the environment has high solar radiation, causing significant drying of the soil in the vegetated base installed on the bare ground within the frame.



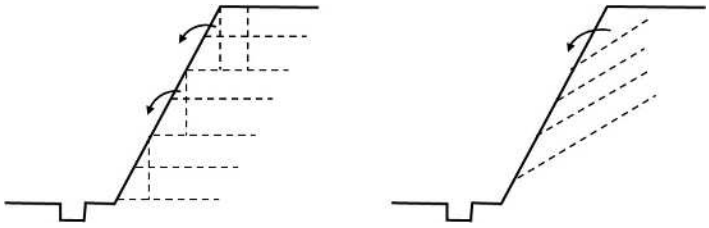
(a) Immediately after construction (October 24, 2006) (b) 13 years after construction (November 24, 2019)

Figure 6.1.2 Vegetation recovery status at shotcrete crib works construction sites as revegetation foundation works

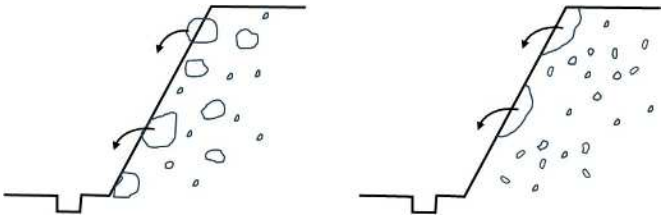
On slopes where the rock mass has been weakened by weathered rock or long slopes, partial collapse patterns (surface stripping type collapse) as shown in Figure 6.1.3 occur. For such collapse patterns, the frame itself is used

to counteract them by employing a cross-section with a frame size of 200×200 or larger.

Furthermore, Table 6.1.3 shows the guideline specifications for shotcrete crib works applied against rockfalls and small collapses.



(a) Collapses occurring along bedding planes or joints



(b) Rockfalls and surface collapses

Figure 6.1.3 Failure modes addressed by shotcrete crib works alone (FFS, 1983)

Table 6.1.3: Guidelines for Shotcrete crib works Specifications for Rockfall and Small-scale collapses Countermeasures(Created with reference to FFS, 2008)

Performance Category	Purpose of Application		Frame Size (mm) (b×h)	Frame Spacing (m) (Vertical Rib × Horizontal Rib)	Remarks
Slope Control Works	Slope Stability	Rockfall and Small-scale collapses Countermeasures	200×200 ～ 300×300	1.2×1.2 ～ 2.0×2.0	Use as standalone frame structure



Figure 6.1.4: Example of shotcrete crib works

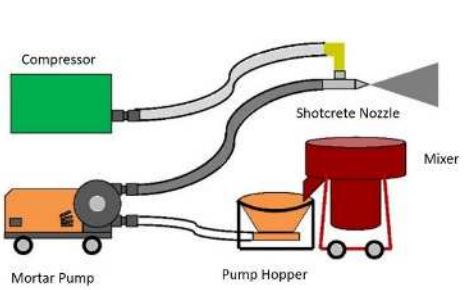
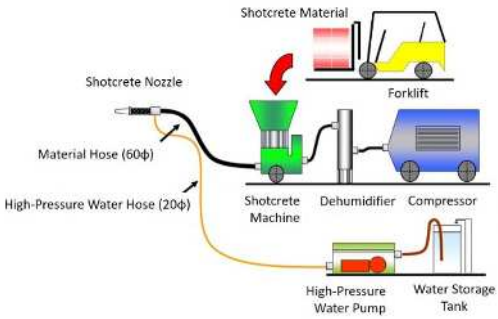


Figure 6.1.5: Installation of Wire Mesh Formwork in shotcrete crib works

Table 6.1.4 compares dry-mix and wet-mix shotcrete methods. The “wet-mix shotcrete method” is the most commonly used standard method because it produces less rebound, allows easier quality control of the mortar compared to the dry-mix method, and enables the construction of higher-strength, higher-quality retaining walls. Furthermore, the weakness of the wet shotcrete method—conveying distance—has been addressed. Recent improvements in hoses and conveying pumps have enabled systems capable of handling distances equivalent to

400 meters horizontally. Consequently, the wet shotcrete method can now be applied to most slopes.

Table 6.1.4 Comparison of Wet-Spray and Dry-Spray Methods (Comparison as of 2022)

	Wet-Spray Method	Dry-Spray Method
Conceptual Diagram		
Overview	A method where materials, including water, are pre-mixed and then sprayed.	A method where sand, cement, etc., are dry-mixed beforehand, then pumped through a sprayer and mixed with water at the nozzle position before spraying.
Mix Control	Quality control is easy since mixing occurs at the on-site plant.	Quality control depends on the nozzle operator's skill since materials delivered via separate systems are mixed at the nozzle tip.
Pump Distance	Standard is within 100m horizontally and 45m vertically. However, recent improvements in hoses and pump systems have enabled systems capable of handling distances equivalent to 400m horizontally and 130m vertically※1.	Horizontal: 300m or less, Vertical: 150m or less※2
Construction Capacity	Approx. 0.5–1.0 m³/h	Approx. 1.0–1.5 m³/h
Spray Thickness	Max. approx. 30 mm	Max. approx. 150 mm
Rebound	Low	Generally high
Post-Construction Cleaning	After completely discharging material from the delivery hose, water washing of the hose is required.	Material in the delivery hose can be instantly discharged using compressed air. No water washing required.

(Additions to Table 1-1-1 on p. 2 of the *Report of the Research Committee on Evaluation Methods for Workability and Quality in Dry-Mix Shotcrete* (JCI,2015))

※1: SLP Shotcrete Method, Sintec Co., Ltd. (http://www.symtec.co.jp)

※2: Repair and Seismic Reinforcement Technology Using Dry-Mix Shotcrete Method, Forum on Repair and Reinforcement of Concrete Structures 2019, August 7, 2019

References

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Shimoeda, H. (2019): Repair and Seismic Retrofitting Technologies Using the Dry-Spray Method with Polymer-Cement Mortar, Forum on Repair and Retrofitting of Concrete Structures 2019 (Aomori Venue, August 7, 2019)

6. 2 Key Issues and Actions to Take in Internationalization

6. 2. 1 Japan-Specific Countermeasure Technologies

(1) Issues

The shotcrete crib works, commonly adopted in Japan as a slope stabilization technique, is a uniquely Japanese slope protection technology developed in the late 1970s (late Showa Era 40s). It applies overseas spraying technology to slope disaster prevention, specifically as a method of spraying onto a grid-like frame structure. While recent construction projects have aimed to promote this technology overseas, its recognition outside Japan remains limited, except in certain regions like Taiwan and Hong Kong. Consequently, to gain adoption internationally, the advantages of the shotcrete crib work method must be demonstrated, and its differences from the commonly used “sprayed concrete” techniques overseas must be clearly explained and clarified.

(2) Explanation

Table 6.2.1 compares “shotcrete works” and “concrete crib works”, as well as “shotcrete crib works” and “cast-in-place concrete crib works”.

a) Comparison and Characteristics of Shotcrete Works and Concrete Crib Works Construction

Shotcrete works are commonly used overseas as a slope protection method for stable slopes, aiming to prevent weathering, erosion, and small rockfalls. However, it has drawbacks such as being unable to accommodate landscape or environmental considerations and being prone to collapse due to earthquakes or deterioration (Figures 6.2.1 and 6.2.2). Furthermore, shotcrete cannot be applied on slopes with abundant surface water or seepage. It also cannot be applied in cold regions where freeze-thaw effects are significant, necessitating an assessment of applicability based on regional characteristics. For these reasons, its adoption has declined in Japan in recent years.

On the other hand, concrete crib works offer higher strength and durability than shotcrete works; in addition to preventing erosion and small rockfalls, they also serve as a foundation for greening and help prevent small-scale collapses. Furthermore, by combining it with drainage works, such as installing highly permeable materials like round stones within the frame, it can be adapted for slopes with abundant surface water or seepage.



Figure 6.2.1 Examples of Damage to Shotcrete works Caused by Earthquakes



Figure 6.2.2 Case Study of Damage to Aging Shotcrete works

Table 6.2.1 Comparison of "shotcrete works" and "concrete crib works" with "Cast-in-Place concrete crib works" and "shotcrete crib works"

Item		Shotcrete works	Concrete crib works	
			Cast-in-place concrete crib works	shotcrete crib works
Function	Preventing weathering and erosion of slopes	◎	○～△ Effect varies depending on treatment method within frame	
	Preventing small rockfalls	◎	◎	
	Greening foundation works	×	◎	
	Small-scale collapse prevention	×	◎	
Characteristics	Durability	○	◎	
	Seismic resistance	-	◎	
	Landscape/environmental harmony (Greenability)	×	○ Greenery within Frame	
	Base condition Verifiability	×	○ Effect varies depending on treatment method within frame	
	Workability	◎	× Difficult formwork installation, reinforcement placement, and concrete pouring on steep slopes, uneven surfaces, and at heights	○
	Quality control ease	×	△ ・Insufficient bond between the natural ground and the concrete formwork ・Overseas, there are cases where reinforcement, such as main steel bars, does not meet structural specifications	◎
	Material procurement ease	◎	◎	△ Imports from Japan
	Construction machinery availability	◎ Recently, more companies overseas own wet-spray machines, which are common internationally	◎	○ Recently, more companies overseas own wet-spray machines, which are common internationally
	Response to water seepage points	× Not suitable for locations with water seepage points or high seepage volume	△ Combine with measures like seepage countermeasures	
	Handling freezing/thawing areas in cold regions	×	△ Caution required if regulations exist based on accumulated cold hours, etc.	△ Caution required if regulations exist based on accumulated cold hours, etc.

◎: Optimal ○: Suitable △: Caution required ×: Unsuitable —: Not applicable

b) Comparison and Characteristics of Cast-in-Place Concrete Crib Works and Shotcrete Crib Works

In Japan, cast-in-place concrete crib works are estimated to have been adopted around the late 1940s. However,

they faced issues such as insufficient bond between the natural ground and the concrete wall, and difficulties in installing formwork and placing concrete on steep slopes or high elevation slopes. Consequently, shotcrete crib works, which combine sprayed concrete technology with concrete crib works construction, were developed in 1950. Shotcrete crib works are constructed by installing wire mesh formwork on a slope, placing reinforcement, and then spraying concrete as shown in Figure 6.2.4. This method facilitates material placement and concrete placement on steep slopes, high-elevation slopes, and uneven slopes. and the ease of mortar quality control through the use of the wet shotcrete method. Consequently, wet shotcrete crib works is now the mainstream method in Japan. However, when adopting shotcrete crib works overseas, factors such as the need to procure materials like formwork and wet shotcrete construction equipment from Japan must be considered.

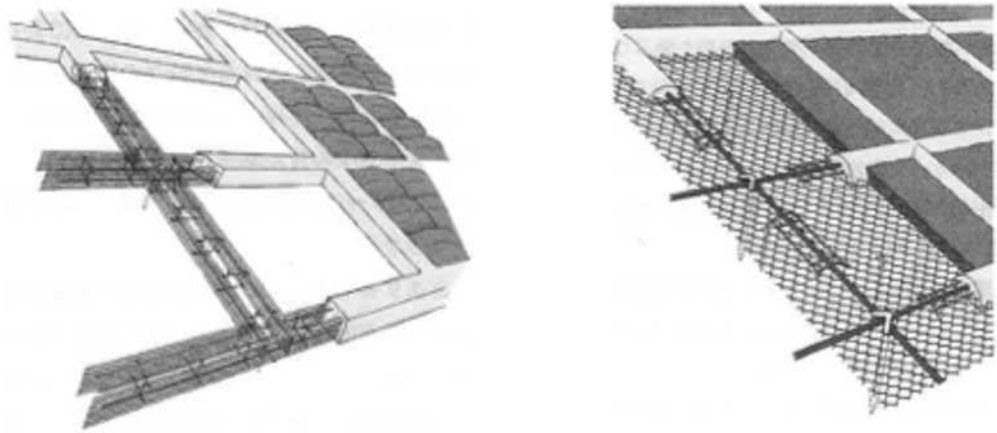


Figure 6.2.3 Example of Wire Mesh Formwork in Shotcrete crib works(JRA, 2009)



Figure 6.2.4 Spray Application Work in Progress

(3) Actions to Take

Explain how shotcrete crib works is a superior method compared to conventional shotcrete works. If wet-mix spraying equipment or wire mesh formwork cannot be prepared, propose using dry-mix shotcrete crib works or cast-in-place concrete crib works as alternatives. In such cases, strict management of the finished form must be implemented, including adherence to structural detail standards.

References

JRA (2009): *Guidelines for Highway Earthwork: Cut Slope Protection and Slope Stabilization*, Japan Road Association (JRA), 570p.

6. 2. 2 Lack of Logical Basis for Simplified Design Calculations

(1) Issues

When using shotcrete crib works alone as slope stabilization measures, simplified design methods targeting small-scale failures up to approximately 1 m in depth have been proposed. However, these simplified design methods are based on experience and lack a logical foundation considering soil properties and slope failure prevention mechanisms. This constitutes an issue when adopting them overseas.

(2) Explanation

1) Design Method for Shotcrete crib works as Erosion Control and Revegetation foundation works

Although omitted in the latest standards, design calculations were performed to determine the specifications for shotcrete crib works at that stage, and they were documented in older standards (FFS, 1983). This design method is introduced below.

This section presents the method for calculating the steel area of horizontal rib for shotcrete crib works used as slope protection works (for weathering prevention and as revegetation foundation works) and the concept for intersection displacement anchorage (FFS, 1983). This design method verifies the strength of the concrete frame itself when materials such as backfill or sandbags are placed within the frame (Figure 6.2.5).

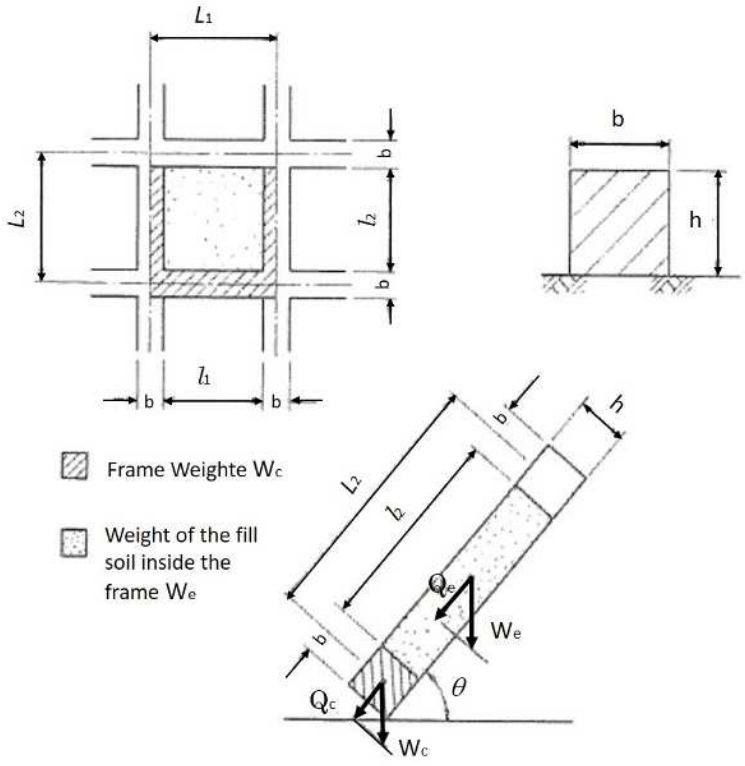


Figure 6.2.5 Concept of Loads Within Frame (FFS, 1983)

(i) Frame Considerations

(a) Calculation of Load *W*

$$W = W_c + W_e$$

Here, W_c: Weight of vertical and horizontal rib per span (1000kgf/span)

W_e: Weight of backfilled soil in the frame's bare area per span (1000kgf/span)

$W_c = (L_1 + l_2) \times b \times h \times \gamma_c$

$W_e = l_1 \times l_2 \times h \times \gamma_e$

L_1, l_1, l_2, b, h : Variables related to frame dimensions (see Figure 6.2.5)

γ_c: Unit weight of mortar or concrete (including reinforcement)

γ_e: Unit weight of backfill soil (γ_c' for mortar or concrete)

(b) Calculation of Uniformly Distributed Load ω on the Horizontal Rib

The load Q (1000kgf/span) acting per span is calculated using the following equation. (In reality, since the frame is in close contact with the ground, adhesion force should be considered; however, for design purposes, this is ignored for safety.)

$$Q = W \cdot \sin\theta \quad (1000\text{kgf/span})$$

Here, θ: Slope angle of the frame (see Figure 5.10)

To calculate the uniformly distributed load ω (tf/m) acting on the crossbeam as the distributed load,

$$\omega = Q / L$$

Here, L: Crossbeam span length (m) (= L_1)

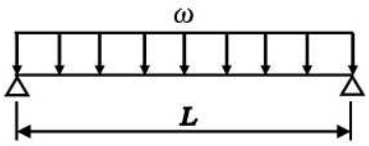


Figure 6.2.6 Load Conditions for a Single Span

(c) Calculation of Bending Moment M on Frame

When considering a simply supported beam subjected to uniformly distributed load,

$$M = \omega L^2 / 8 \quad (1000\text{kgf·m})$$

When considering a continuous beam subjected to uniformly distributed load,

$$M = \omega L^2 / 10 \quad (1000\text{kgf·m})$$

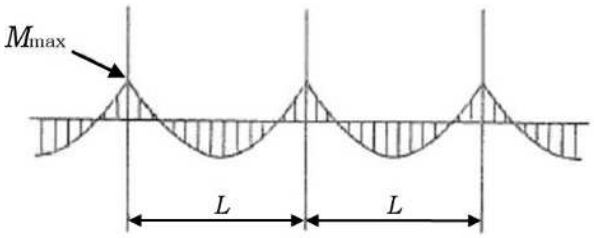


Figure 6.2.7 Bending Moment Distribution for Continuous Burrs

(d) Calculation of Required Reinforcing Steel Area A_s (m²)

$$A_s' = \frac{M}{\sigma_{sa} \cdot \frac{7}{8} \cdot d}$$

Here, d: Effective height of main reinforcing bars (m)

Use A_s such that $A'_s \leq A_s$.

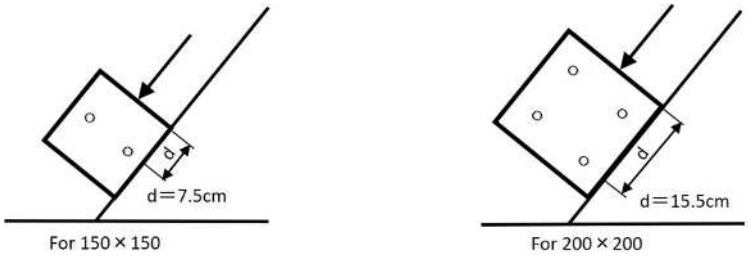


Figure 6.2.8 Example of Reinforcement Arrangement Inside Frames
(Left: Rib cross-section 150x150, Right: Rib cross-section 200x200) (FFS, 1983)

(e) Stress Analysis (σ_s: Reinforcing Steel, σ_c: Mortar or Concrete)

$$\sigma_s = \frac{M}{A_s \cdot j \cdot d}, \qquad \sigma_c = \frac{\sigma_s}{m}$$

Calculate j and m from the reinforcement ratio $p = \mathbf{A}_s / (b \cdot d)$.

Conduct stress analysis assuming $\sigma_s \leq \sigma_{sa}$ and $\sigma_c \leq \sigma_{ca}$.

(ii) Anchor Considerations

(a) Main Anchor

The main anchor is used to prevent the retaining frame from sliding down due to its own weight and the weight of the exposed soil within the frame after shotcrete application. Therefore, it must satisfy the following two conditions:

- Possess shear resistance sufficient for the load of one span of the concrete crib work
- Have a length approximately twice the depth of ground freezing in winter

(b) Auxiliary Anchors

Auxiliary anchors are installed to prevent formwork deformation during formwork setting and to prevent frame deformation after shotcrete application.

The specifications for main anchors and auxiliary anchors shall be determined by referring to Table 6.2.2.

Table 6.2.2 Specifications for Primary and Secondary Anchors in Shotcrete crib works Used for Erosion Control and Greening Foundation Works

Standard Rib Cross-Section and Span (mm)	Main Anchor (mm)	Auxiliary Anchor (mm)
150×150 1150×1150	D13×500	D10×300
150×150 1500×1500	D16×500	D10×300
200×200 1200×1200	D16×750	D10×400
200×200 1500×1200	D16×750	D10×400
300×300 2000×2000	D19×1000	D13×600

Note: Anchors shall be deformed bars SD30.

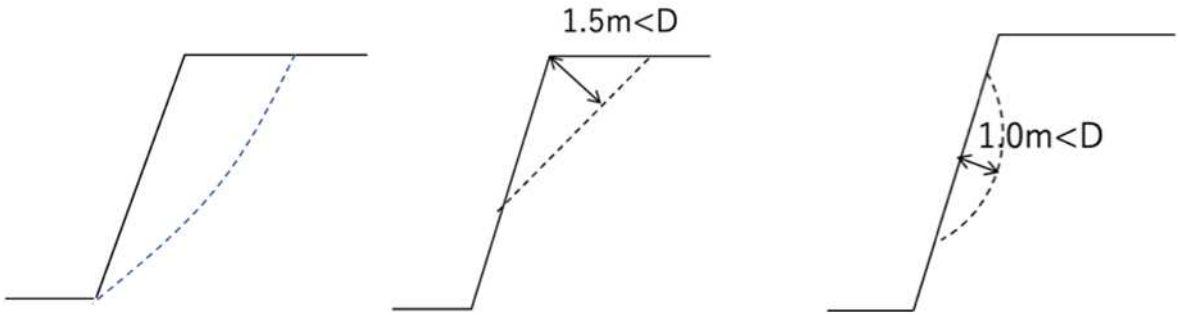
2) Design Method for Shotcrete crib works to Prevent Small-scale Collapses

Tables 6.2.3 and 6.2.4 show simplified design methods for collapse types that can be addressed by the retaining structure alone. Originally used as erosion control structures or revegetation foundation works, shotcrete crib works have come to be employed as restraint measures. Consequently, these simplified design methods were empirically developed. Therefore, issues exist regarding the external forces acting on the concrete frames and the calculation models.

On the other hand, while current design calculation methods have issues such as requiring soil property constants to be set, their effectiveness has been demonstrated in practice (e.g., NILIM, 2011) by consistently achieving sufficient small-scale collapse prevention effects, including during earthquakes. This must be explained when introducing the method.

Note that the concrete crib works alone cannot address the following failure patterns, so caution is required.

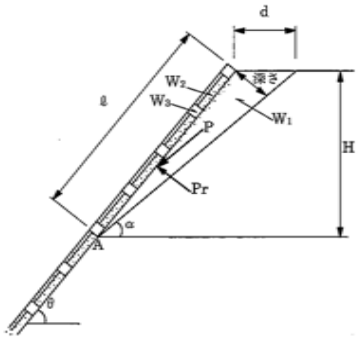
- Failure extending from the toe to the crown (Figure 6.2.9(a))
- Failure exceeding 1.5 m in depth when originating from the toe (Figure 6.2.9(b))
- Failure exceeding 1.0 m in depth when originating from the mid-section (Figure 6.2.9(c))



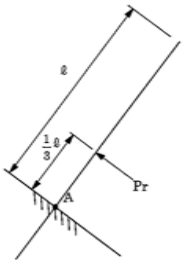
(a) Total collapse **(b) Collapse of the toe exceeding 1.5 meters in depthm** **(c) Collapse of the intermediate section exceeding 1 meter in depth**

Figure 6.2.9 Failure Modes Not Addressable by Concrete crib works Alone

Table 6.2.3 Simplified Design Calculation Method for Shotcrete crib works - Collapse from the Slope Foot

<table>
<tr><th>Case</th><th colspan="3">Collapse from Sloping Shoulders</th></tr>
<tr><td>Load Setting</td><td colspan="3">Assumed to be the weight of the sliding soil mass, infill material, and the self-weight of the vertical and horizontal ribs.</td></tr>
<tr><td>Applied Load</td><td colspan="3">(1) The load P to be considered shall be the component of the load in the direction of the slip plane multiplied by the safety factor increase ΔF_s.
(2) The load P_r acting on the vertical rib is the component of P perpendicular to the frame. Its point of application is at one-third of the soil block's length l.
(3) The design load P_d shall be calculated by multiplying P_r by the load factor.</td></tr>
<tr><td>Rib Cross-section Analysis</td><td colspan="3">Treat part of the vertical rib as a cantilever beam. Treat the intersection point between the vertical rib and the slip line as a fixed point. Verify the stability of the vertical rib members against the bending moment generated in the vertical rib.</td></tr>
<tr><td rowspan="2">Load Model</td><td rowspan="2">

Load Model



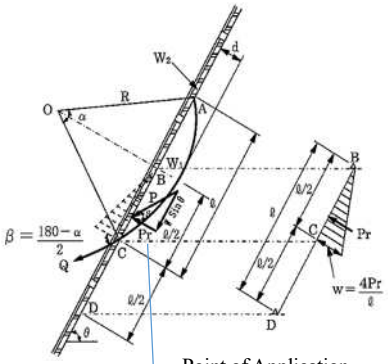
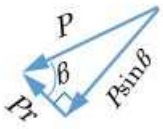
Point of Application of Loads and Support Conditions</td><td>Explanation</td><td>In the “Load Model” diagram on the left, the load acting on the vertical rib is the vertical load Pr acting on the vertical rib of the shear force P.
Here,
$(W\cdot\cos\alpha\cdot\tan\phi+c\cdot l+P)\ /W\cdot\sin\alpha\geqq F_s$
If we set $(W\cdot\cos\alpha\cdot\tan\phi+c\cdot l)\ /W\cdot\sin\alpha=1.0$, then
$1.0+P/W\sin\alpha=1.0+\Delta F_s$
$\therefore\ P=\Delta F_s\cdot W\cdot\sin\alpha$
$Pr=P\cdot\sin(\theta-\alpha)$
W : Sliding weight (sliding soil mass + infill material + frame)
α : Slope angle of the sliding surface
θ : Slope angle of the slope
ϕ : Internal friction angle of the sliding mass
c : Cohesion of the sliding mass
l : Length of the sliding surface

In the figure “Point of Application of Loads and Support Conditions” shown on the left, the position where the sliding load acts is defined as one-third of the length from point A, where the retaining wall is integrated with the ground that does not collapse. Calculate the bending moment in the retaining wall as a cantilever beam structure with point A as the fixed end.
$M_{max}=\gamma_f\cdot P_r\cdot l/3$
Here, γ_f : Load factor(=1.2)

Verify that the tensile stress in the vertical rib's reinforcing bars and the compressive stress in the mortar remain within the allowable stress limits.</td></tr>
<tr><td>Issues</td><td>1) Assuming a safety factor of 1.0 for the current slope, calculate the load acting on the vertical rib P_r.
It is not common practice overseas to perform stability analysis assuming a safety factor of 1.0 for existing slopes, so an explanation of the rationale for setting it to 1.0 is necessary. Furthermore, the effects of c and φ on the sliding soil mass have not been considered.
2) The axial component of the sliding force in the vertical rib is not considered
3) The length of the vertical rib below the intersection point—which is required to define the fixed end at the intersection of the vertical rib and the sliding surface—is not clearly specified.
4) The effect of the horizontal frame is unclear</td></tr>
</table>

(Compiled from pp. 38–42 of JSPA(2013))

Table 6.2.4 Simplified Design Calculation Method for Shotcrete crib works—Collapse from the Middle of the Grids

<table>
<tr><th>Case:</th><th colspan="3">Collapse from the middle of the grids</th></tr>
<tr><td>Load setting</td><td colspan="3">Assumed to be the dead weight of the sliding soil mass and infill material.</td></tr>
<tr><td>Applied load</td><td colspan="3">(1) The load P to be considered is the component of the load in the direction of the slip plane multiplied by the safety factor increase ΔFs.
(2) The load P_r acting on the vertical rib is defined as the component of P in the direction perpendicular to the frame.
(3) The design load P_d shall be P_r multiplied by the load factor.</td></tr>
<tr><td>Rib Cross-section Analysis</td><td colspan="3">Part of the vertical rib is treated as a simple beam, and the applied force is replaced with a triangular distributed load where the force is maximum at the midspan and zero at the intersection points.</td></tr>
<tr><td rowspan="2">Load Model</td><td rowspan="2">
Load Model
Point of Application of Loads and Support Conditions
The Wsinβ in the original figure is incorrect.

The Correct Relationship Between P and Pr</td><td>Explanation</td><td>Assume an arc-shaped slip occurs where the depth d is less than 1.0m. The resultant earth slip force P is calculated as follows.
$P = \Delta F_s \cdot \Sigma W_i \cdot \sin \alpha_i$
However, considering the safety margin,
$P = \Delta F_s \cdot W \cdot \sin \theta$
is used for calculation.
This combined earth mass sliding force acts tangentially (perpendicular to line OC in the left diagram) at the point where the arc intersects the slope (point c in the left diagram).
Considering that the vertical load P_r acts perpendicular to the longitudinal rib of this soil block sliding force, distributed triangularly (with the maximum value w of the triangular distribution acting at the center of the span), the bending moment generated in the vertical rib is calculated.
$P_r = P \cdot \cos((180-\alpha)/2)$
$\omega = 4 \cdot P_r / l$
$M_{max} = (4 \cdot \gamma_f \cdot P_r \cdot l) / (9 \cdot \sqrt{6})$
Here, γ_f : Load factor (=1.2)
Verify that the tensile stress in the reinforcing bars within the vertical rib and the compressive stress in the mortar remain within the allowable stress limits.</td></tr>
<tr><td>Issues</td><td>1) The resultant force of the circular slide is evaluated as $W\sin\theta$, failing to properly assess circular slide.
2) The frame weight considered for the case of collapse starting from the sloping shoulder has not been taken into account as a design load.
3) The force along the vertical axis in the component diagram showing the relationship between P and P_r is incorrect. (See enlarged diagram.)
4) Point B, the midpoint of the arc-shaped collapse, is assumed to be a support point (a hinged support allowing only rotation) for a simply supported beam (between B and D).
5) The treatment of the slope-direction component of the sliding force is unclear.
6) The effect of the horizontal rib is unclear.</td></tr>
</table>

(Compiled from pp. 38–42 of JSPA(2013))

(3) Actions to Take

The current method, while lacking strong logical justification, possesses minimal verification functions such as excluding extremely small rib cross-sections. It is introduced as a simplified design approach based on its extensive adoption in Japan without reported issues. For specific design methods, refer to the “Design and Construction Guidelines for Concrete crib works ” (JSPA, 2013).

References

FFS (1983): *Guidelines for Design and Construction of Free Frame Metho*d, Free Frame Society (FFS), 112p.

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JSPA (2006): *Design and Construction Guidelines for Concrete Crib Works for slope*, Japan Slope Protection Association (JSPA), 85p.

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NILIM (2011): Quick Report on Damage to Infrastructures by the 2011 off the Pacific coast of Tohoku Earthquake, Technical Note of National Institute for Land and Infrastructure Management (NILIM), No.646, pp.481.

6. 3 Design Considerations

(1) Preliminary Slope Stability Assessment (Geology, Groundwater, etc.)

The condition for adopting a concrete crib works is that it must prevent the occurrence of “collapses that cannot be addressed by the concrete crib works alone” as shown in Figure 6.2.9. Therefore, ground conditions must be confirmed through preliminary ground investigations (soil properties, geology, groundwater, collapse history, etc.). For the design of concrete crib works incorporating restraint measures such as slope reinforcement works or ground anchors, refer to Chapter seven for implementation.

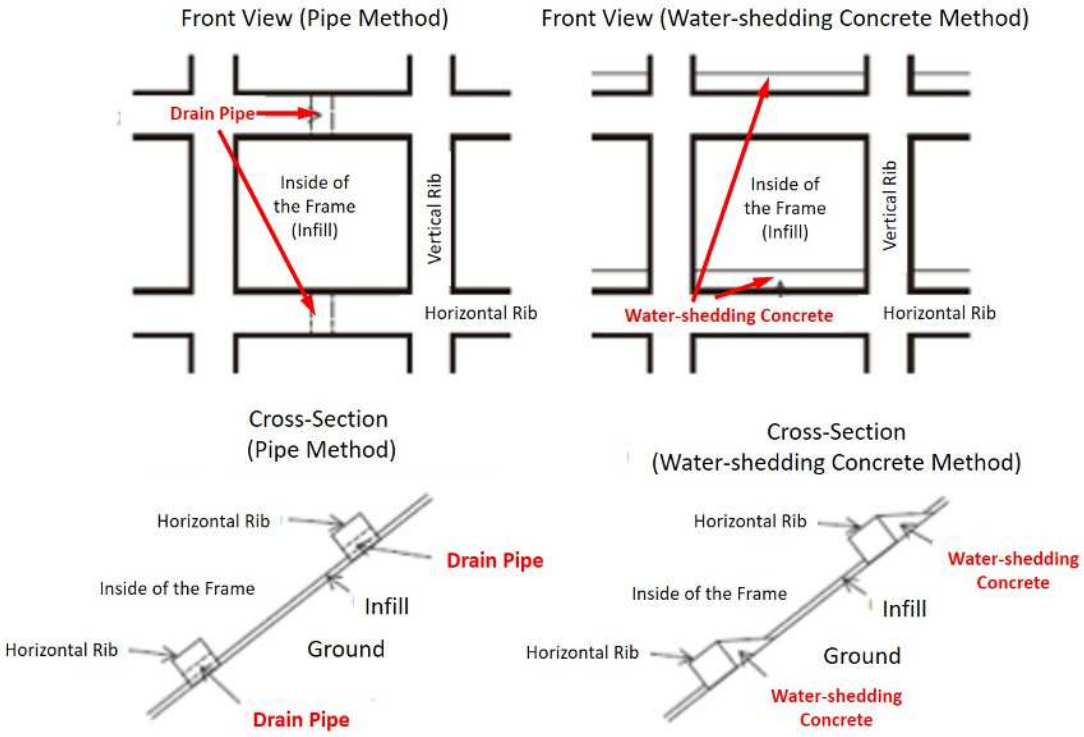
(2) Treatment of the Lower Edge of the Concrete Frame

While shotcrete crib works can omit foundation work, setting the lower edge at an intermediate point mid-slope poses a risk of the entire wall sliding down during torrential rains. Therefore, when placing the lower edge mid-slope, it is preferable to position it at points of slope break, such as concave slope break lines.

(3) Drainage Measures

For concrete crib works, when the interior is open (such as when implementing vegetative works or vegetative base works), drainage works for shallow groundwater removal are generally not implemented.

Furthermore, regardless of whether the interior is closed, rainwater or other water stagnating within the frame may cause deterioration of the vegetation or the horizontal rib. Therefore, for drainage within the frame, either a drain pipe with a diameter of approximately 50 mm must be installed inside the horizontal rib (pipe method, Fig. 6.2.10(a)), or a triangular water-shedding measure must be implemented on the upper part of the horizontal rib (water-shedding method, Fig. 6.2.10(b)).



(a) Drain Pipe Method **(b) Water-shedding Concrete Method**

Figure 6.2.10 Example of Frame-Interior Drainage

(4) Shotcrete Materials

Shotcrete crib works shall primarily use mortar spraying. This is because, compared to concrete, it produces less rebound, reduces the risk of rebounded material being entrained into newly sprayed material, and minimizes the likelihood of honeycombing occurring in the gaps between reinforcing bars and formwork.

The design strength standard for sprayed mortar is currently set at 18 N/mm², but prior to 2005, it was 15 N/mm². Developing countries also adopt 18 N/mm² as the standard design strength. While this may result in strength variations, even a reduction to around 15 N/mm² does not significantly compromise its functionality as a remedial measure.

(5) Design Using Locally Available Reinforcing Bars

Since the specifications of reinforcing bars available in Japan and locally (such as standard strength, diameter types, and the relationship between diameter and cross-sectional area) differ, design must be based on the specifications of locally available reinforcing bars.

7. Design of Ground Anchors

7. 1 Overview of Ground Anchors

Figure 7.1.1 shows a schematic diagram of ground anchor works. This technique stabilizes slopes by inserting anchor tendons into boreholes, injecting grout to form the bond length within the stable stratum, and subsequently applying tension to the tendons from the ground surface. In Japan, this technique is extensively utilized as a countermeasure against reactivated slow-moving landslides and as a preventive measure against failures on steep slopes and cut slopes. As of 2012, a cumulative total of 1.68 million anchors had been installed for slope stabilization.

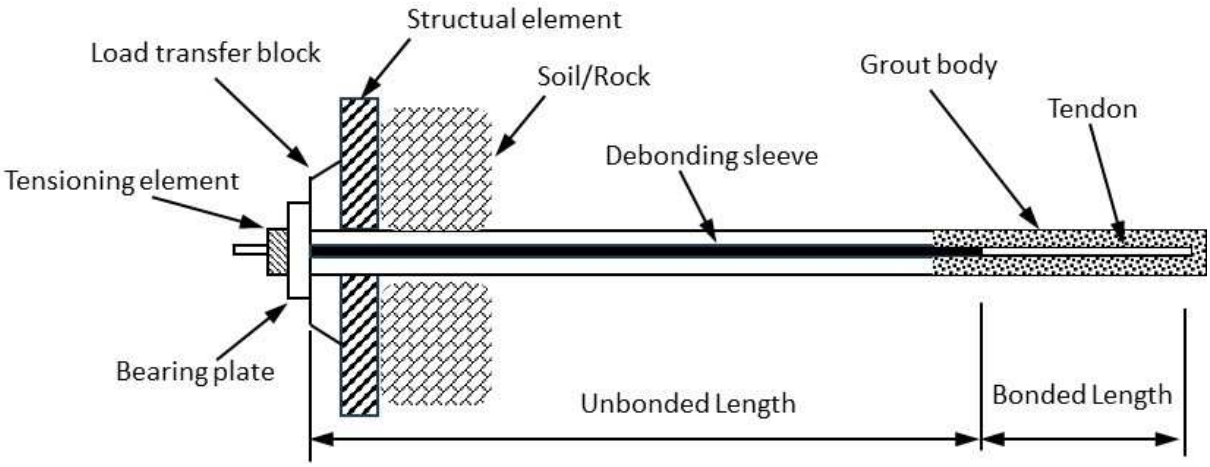


Figure 7.1.1 Schematic of ground anchor structures (BS EN 1537:1999)

Hobst & Zajic (1983) comprehensively describe the design, testing, and construction methods for ground anchors as slope stabilization measures, indicating that these anchors were already utilized in Europe and North America during that era. Ground anchors for slope stabilization are also included in the technical standards for landslide mitigation published by the Indian Roads Congress (IRC, 1995). While the latest design philosophy is outlined in Eurocode 7, Section 8, it is based on performance-based design (limit state design methods), and thus lacks explicit structural calculation formulas or specific material properties. An excerpt from the beginning of the design formulas for ground anchors specified in Eurocode 7, Section 8 is shown below.

＜Ultimate limit state (ULS) design force to be resisted by the anchors＞

$E_{ULS,d} \leqq R_{ULS;d}$ ・・・Eq.8.1(EC7 Section 8)

$E_{ULS,d} = max(F_{ULS,d}; F_{Serv,d})$ ・・・Eq.8.2(EC7 Section 8)

$F_{ULS,d} = \gamma_{Serv} F_{Serv;k}$ ・・・Eq.8.3(EC7 Section 8)

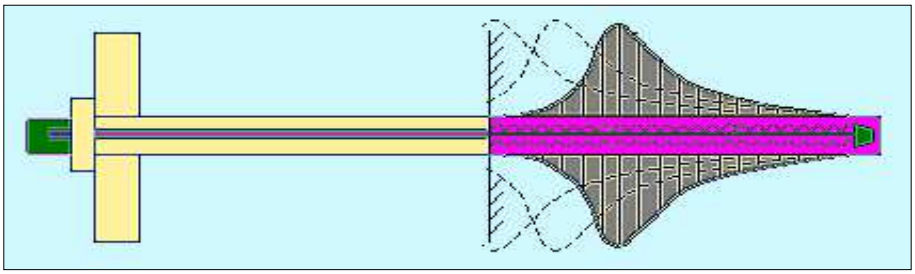
$R_{ULS;d}$ = Design value of the resistance of an anchor complying with ULS criteria
$E_{ULS,d}$ = ULS design force to be resisted by the anchor
$F_{ULS,d}$ = Design value of the force required to prevent any ULS in the supported structure.
$F_{Serv,d}$ = design value of the maximum anchor force (expected within the design life of the anchor), including effect of lock off load, and sufficient to prevent a SLS in the supported structure.
γ_{Serv} = partial factor

Thus, while ground anchor design in Europe and North America has shifted to performance-based specifications, prescriptive (specification-based) design is still practiced in Japan as of 2023, and the contents of this chapter are

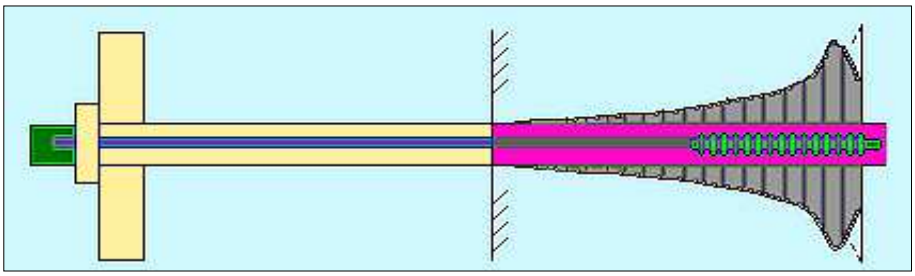
based on these prescriptive standards. For field testing methods to determine anchor design parameters, an international standard has been established as ISO 22477-5.

While temporary anchors and rock anchors dominate the market in Europe and North America, the application of anchors in Japan has expanded since the 1990s as a permanent countermeasure for slope failures, even on slopes consisting of highly weathered soil or colluvial deposits. Consequently, a wide variety of anchor products have been developed to meet these diverse needs. Furthermore, even within the same category of bonded pre-stressed anchors, they are classified into two main types: compression-type and tension-type. Bottle-shaped pre-stressed anchors are also available (see Figure 7.1.2). These different types are selected based on the specific properties of the anchorage ground. Even for standard cementitious grouting, many products feature enhanced corrosion protection, and some offer even higher durability by utilizing resin-based grouts with excellent acid and heat resistance.

(a) Tensile stress type bonded pre-stressed anchor



(b) Compressive stress type bonded pre-stressed anchor



(c) Bottle-shaped pre-stressed anchor

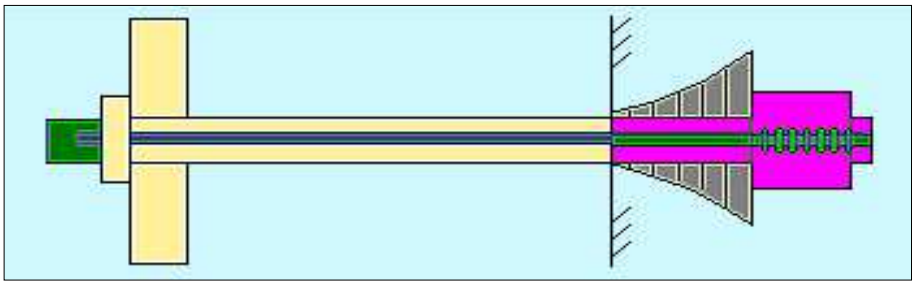


Figure 7.1.2 Anchor types and stress distribution in the bonded zone

References

Hobst,L., Zajic, J. (1983): *Anchoring in rock and soil*, developments in geotechnical engineering 33, Elsevier, 570p.

IRC (1995): *State of the Art: Landslide correction techniques*, Special report 15, IRC highway research board New delhi

ISO 22477-5:2018 Geotechnical investigation and testing — Testing of geotechnical structures — Part 5: Testing of grouted anchors

BS 8081:1989 British Standard code of practice for Ground Anchorages. BSI, 2 Park St, London.

BS EN1537:1999 Execution of special geotechnical work – Ground Anchors.

BS EN 1537:2000 European Standard for Execution of special geotechnical work - Ground Anchors.

BS EN1537:2013 Execution of special geotechnical work – Ground Anchors.

FIP: 1996 Recommendations for the design and construction of prestressed ground anchorages.

FIP: 1986 Corrosion and corrosion protection of prestressed ground anchorages, Thomas Telford Ltd, London EC1P 1JH, England.

Eurocode 7 Section 8-Ground anchors

<References for Root Pile>

Lizzi, F. (1983): The Reticolo Di PaliRadice (Reticulated Root pile) for the improvement of soil resistance, physical aspects and design approaches-VIII ECSMFE, Helsinki, pp. 521-524.

7. 2 Key Issues and Actions to Take in Internationalization

7. 2. 1 The Lack of Mechanical Rigor in the Methodology for Calculating Required Stabilizing Force Widely Used in Japan

(1) Issues

In Japan, both the denominator-term and numerator-term approaches in slope stability analysis formulas are used to calculate the required stabilizing force for ground anchors. Although adding the driving force reduction effect to the numerator lacks strict mechanical rigor, it remains the mainstream approach in Japan from the perspective of "conservative design." Overseas, however, this "conservative design" approach might be perceived as synonymous with "overdesign." Conversely, design software in Europe and North America typically places the driving force reduction effect in the denominator term. Furthermore, some of these software programs do not account for the shear resistance increase effect, while others evaluate the anchor effect using entirely new methodologies distinct from those applied in Japan.

(2) Explanation

1) Japan's Calculation Method for the Required Stabilizing Force Lacking Mechanical Rigor

Figure 7.2.1 illustrates the concept of anchor component forces on a slip surface, as presented by Hobst & Zajic (1983) in *Anchoring in Rock and Soil*. In Japan, ground anchors began to be applied as slope stabilization measures in the late 1960s, and this identical concept of anchor component forces was introduced. This represents the classical approach to evaluating anchor component forces.

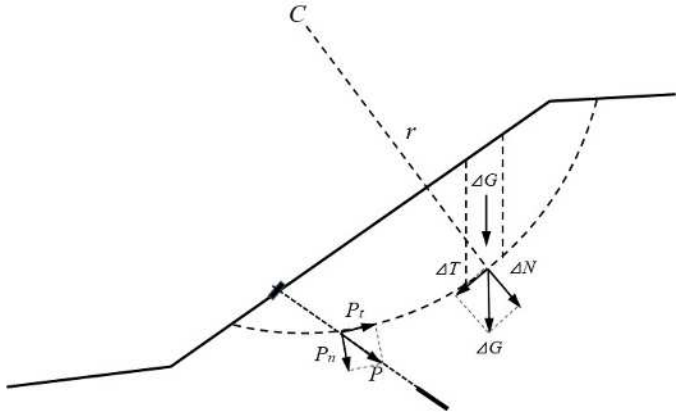


Figure 7.2.1 Component forces of a ground anchor according to Hobst & Zajic (1983)

The shear resistance increase effect occurs when the normal component of the anchor force increases the effective earth pressure on the slip surface, thereby increasing shear resistance, and it always serves as a numerator term in slope stability analysis formulas. On the other hand, the driving force reduction effect directly inhibits sliding through the tangential component of the anchor force; mechanically, it is correct to place this component in the denominator term of the stability analysis formula. Consequently, adding the driving force reduction effect to the numerator term lacks mechanical rigor. Hobst & Zajic (1983) adopted a mechanically correct formula that places the driving force reduction effect in the denominator term. In Japan, until the 1980s, the mainstream approach was to add the stabilizing force of restraint measures—such as steel pipe piles and ground anchor works—to the numerator term of the stability analysis formula. As a result, the method of placing the anchor's driving force reduction effect in the numerator term has continued to be used by convention and remains the mainstream today.

2) Mechanical Interpretation of the Approach Placing the Driving Force Reduction Effect in the Numerator Term

All solid bodies, such as desks and walls, possess compressive and shear strengths. However, these strengths are intrinsic properties (scalars) of the solid material and remain latent. Similarly, in the limit equilibrium method, the shear strength in the numerator term of the stability analysis formula is a latent scalar; therefore, a force of that exact magnitude does not actually manifest on the slip surface of a non-sliding slope. It is a fundamental principle of the limit equilibrium method that the shear strength of the slip surface divided by the factor of safety F represents the mobilized force (shear force). According to the laws of mechanics, shear strength (a scalar) and force (a vector) cannot be added directly. Consequently, the Japanese approach of adding the driving force reduction effect to the numerator term lacks strict mechanical rigor.

Placing the driving force reduction effect in the numerator term of the stability analysis formula results in a required stabilizing force that is approximately 10% to 20% larger than when it is placed in the denominator term. Based on this outcome, the concept of "conservative design" has been adopted in Japan. In overseas projects, however, this cannot be used to justify proposing a mechanically incorrect methodology.

If a partial safety factor for this 10% to 20% difference is deemed necessary, the partial safety factor should be adopted only after clarifying its engineering rationale; however, no such clear rationale exists at present.

3) Design software in Europe and North America with calculation methods for the required stabilizing force different from Japan

In Europe and North America, while the shear-resistance increasing effect is occasionally considered in stability analysis formulas for rock slopes with linear slip surfaces, the mainstream approach for circular slip surfaces does not account for this effect. SLOPE/W, a prominent design software developed in the United States, supports the design of both ground anchors and soil nailing. However, its technical manual treats the anchor force as a concentrated load applied in the installation direction on the slip surface, accounting only for the driving force reduction effect by subtracting it from the denominator term (driving force).Although SLOPE/W supports various stability analysis methods, the recommended General Limit Equilibrium (GLE) formulation indirectly accounts for the shear-resistance increasing effect because the normal force N on the slip surface is dynamically altered by the concentrated load of the anchor. This approach extends the classical concept of anchor component forces, such as that by Hobst & Zajic (1983), into the general limit equilibrium framework, sharing the same fundamental principle.

On the other hand, the design software "SLOPE" developed by GeoStru in Italy treats the horizontal component R_j of the anchor force acting on the slip surface as the anchor's restraint force. This software adopts an approach that adjusts the direction of this horizontal component to align with the tangential direction of the slip surface, subsequently subtracting this adjusted force from the denominator term (driving force) as the driving force reduction effect. For this directional adjustment, the software utilizes a unique methodology: dividing the horizontal component R_j of the anchor force by the direction cosine $R_j/\cos\alpha$ of the slip surface inclination angle α, expressed as $R_j/\cos\alpha$. This method offers the distinct advantage that the driving force reduction effect increases even when the inclination angle of the slip surface becomes steep, thereby allowing the anchor's full restraining capacity to be effectively exerted.

Thus, while the design methodologies adopted by software programs in Europe and North America vary, each

approach is fundamentally rooted in mechanically correct principles.

The concentrated load method adopted by SLOPE/W can indirectly evaluate the shear-resistance increasing effect by increasing the normal force N on the slip surface when used in combination with rigorous limit equilibrium methods, such as Spencer's method or the General Limit Equilibrium (GLE) formulation. However, these rigorous limit equilibrium methods have distinct disadvantages; for instance, the factor of safety varies depending on the assumed position of a fictitious thrust line that does not actually exist, and design values can differ depending on the specific analysis software used. When utilizing the simplified Bishop or simplified Janbu methods, the classical concept of anchor component forces by Hobst & Zajic (1983) remains highly practical. Meanwhile, because the method adopted by SLOPE in Italy is vastly different from traditional Japanese approaches, its track record remains unverified in Japan, making its immediate adoption difficult.

(3) Actions to Take

Overseas design software fundamentally employs a mechanically correct method that places the driving force reduction effect of ground anchors in the denominator term. The concept of "conservative design," which serves as the rationale for adopting the method lacking mechanical rigor that is currently mainstream in Japan, is nearly synonymous with "overdesign." Amid existing concerns that Japanese technology—while high-quality—is too expensive to adopt, it is highly unlikely that the concept of "conservative design," which inevitably increases construction costs, will be accepted in overseas projects. Therefore, when transferring Japanese slope disaster prevention technology abroad, the objective should be to promote the dissemination of mechanically correct methods. Furthermore, when utilizing European or North American design software commonly used in target countries, it is essential to thoroughly understand that design methodologies differ between software programs and to fully comprehend the specific calculation method being employed before application.

<What Is Required Stabilizing Force?>

The following is a supplementary explanation of the required stabilizing force. In Japan, restraint measures such as steel pipe piling, ground anchor, and shaft works have been widely adopted as countermeasure works for slow-moving landslides. The required stabilizing force is a concept common to these restraint measures; it refers to the value Pt that must be subtracted from the denominator, or the value Ps that must be added to the numerator of the stability analysis equation to achieve the target safety factor through the effectiveness of the restraint measures. Therefore, it differs from the horizontal force used in the design of steel pipe piling or the design anchor force in ground anchor works.

References

Hobst, L., Zajic, J. (1983): *Anchoring in rock and soil*, Development in geotechnical engineering 33.

GEO-SLOPE (2015): Stability modeling with SLOPE/W.

GeoStru (2021): Slope stability analysis, https://www.geostru.eu/blog/2016/06/13/slope-stability-analysis/, Accessed 18 Jun 2021.

7. 2. 2 Applicability of Various Prescriptive Values Adopted as Design Conditions in Japan

(1) Issues

In Japan, design standard values for ultimate bond strength τ are predefined for each ground type; however, determining how to appropriately establish these baseline values in a target country with differing geological conditions poses a significant challenge. Similarly, multiple parameters are unique to Japanese standards, such as the classification of anchors using a two-year service life as the threshold, the assigned safety factors f_s against ultimate pull-out capacity, the reference values for the allowable bond stress of tendons, the minimum unbonded (free) length, and the specific regulations on the fixed anchor (bond) length. Consequently, careful reconciliation and consistency with the existing local standards in the target country are strictly required.

(2) Explanation

(a) Ultimate Bond Strength

Regarding ultimate bond strength, while it is mandatory to determine the actual design values through anchor investigation tests during the detailed design or construction stage, standard baseline values are required for preliminary design prior to that phase. Japan's standard values primarily focus on ground classification targeting soils; however, for rock masses and weathered rocks—which typically serve as the anchorage ground on natural slopes—they only provide broad categorizations such as hard rock, soft rock, and weathered rock (Table 7.2.1).

Table 7.2.2 presents data compiled by Yamada (2009), which details the criteria for the ultimate bond strength of ground anchors specified in the PTI (Post-Tensioning Institute) standard document, Recommendations for Prestressed Rock and Soil Anchors. In this standard, baseline constants are defined by subdividing rock types and conditions; for instance, weathered marl is designated at 0.15 to 0.25 MN/m², soft shale at 0.2 to 0.8 MN/m², and weathered sandstone at 0.7 to 0.8 MN/m². Notably, the ultimate bond strength for weathered marl is set at approximately one-fourth of the value specified for weathered rock in the Japanese Geotechnical Society (JGS) standard (Table 7.2.1). Because the target classification category is explicitly labeled as “ROCK,” these PTI criteria specifically apply to rock anchors.

Table 7.2.1 Ultimate bond strength of ground anchors (Japan's standard baseline values) (JGS, 2000)

Type of Ground			Bond Strength (MN/m²)
Rock	Hard Rock*		1.5 – 2.5
	Soft Rock*		1.0 – 1.5
	Weathered Rock		0.6 – 1.0
	very stiff clay		0.6 – 1.2
Gravel	N value	10	0.1 – 0.2
		20	0.17 – 0.25
		30	0.25 – 0.35
		40	0.35 – 0.45
		50	0.45 – 0.7
Sand	N value	10	0.1 – 0.14
		20	0.18 – 0.22
		30	0.23 – 0.27
		40	0.29 – 0.35
		50	0.3 – 0.4
Cohesive soil			10.0×c (c is cohesion (kN/m²))

A classification unique to Japan based on earthwork workability

Table 7.2.2 Ultimate Bond Strength of Anchors under American Standards (PTI) (Yamada, 2009)

Rock	Average Ultimate Bond Stress-Rock/Grout	
	Mpa	PSI
Granite & Basalt	1.7 – 3.1	250 – 450
Dolomite Limestone	1.4 - 2.1	200 – 300
Soft Limestone	1.0 – 1.4	150 – 200
Slates & Hard Shales	0.8 – 1.4	120 – 200
Soft Shales	0.2 – 0.8	30 – 120
Sandstones	0.8 – 1.7	120 -250
Weathered Sadstones	0.7 – 0.8	100 – 120
Chalk	0.2 – 1.1	30 – 155
Weathered Marl	0.15 – 0.25	25 – 35
Concrete	1.4 – 2.8	200 - 400

Even if the rock type is identical, geological ages and ground formation processes differ between Japan and overseas, leading to variations in geological continuity, homogeneity, and strength. Because the engineering strength ranges corresponding to "hard rock" differ between Japan and overseas, the rock mass strengths assumed for each classification of hard rock, soft rock, and weathered rock in Japan are distinct from those overseas. Therefore, when adopting Japan's standard baseline values (Table 7.2.1), it is necessary to reclassify the rock types of the target country according to Japan's categories, which are inherently defined by earthwork workability (excavatability).

In Japan, the distinction between hard rock and soft rock is determined as shown in Table 7.2.3 below.

Table 7.2.3 Rock classification in earthworks (MLIT, Common Specifications for Civil Engineering Works, March 2011)

<table>
<tr><th colspan="4">Name</th><th rowspan="2">Description</th><th rowspan="2">Summary</th></tr>
<tr><th>A</th><th>B</th><th colspan="2">C</th></tr>
<tr><td rowspan="5">Rock</td><td colspan="3">Rock mass, Boulders</td><td>・Rock mass containing boulders, making excavation difficult and prone to clogging buckets, etc.
・Rock mass and boulders shall have a particle size of 7.5 cm or larger, with rounded ones classified as boulders.</td><td>Soil mixed with boulders, rock mass, crushed rock, and boulder-strewn riverbed</td></tr>
<tr><td rowspan="2">Soft Rock</td><td rowspan="2">Soft Rock</td><td>I</td><td>・Tertiary rocks with weak consolidation.
・Extremely weathered and highly friable.
・Materials that can be separated by hand, with cracks spaced approximately 1–5 cm apart, and Tertiary rocks with good consolidation.
・ Significantly weathered with some discoloration, easily fractured by light impact, prone to separation, with crack spacing approximately 5–10 cm.</td><td rowspan="2">Elastic wave velocity
700 – 2800m/s</td></tr>
<tr><td>II</td><td>・Tuffaceous and firmly consolidated. Significantly weathered along the grain.
・ Fracture spacing of approximately 10–30 cm, allowing separation with light impact; heterogeneous, hard interbedded layers that can be easily separated along bedding planes.</td></tr>
<tr><td rowspan="3">Hard Rock</td><td colspan="2">Medium-hard rock</td><td>Limestone, porous andesite, etc., possessing considerable hardness even if not particularly dense. Not significantly weathered. Hard rock with cracks spaced approximately 30–50 cm apart.</td><td>Elastic wave velocity
2000 – 4000m/s</td></tr>
<tr><td rowspan="2">Hard Rock</td><td>I</td><td>Granite, gneiss, or similar rocks showing no alteration whatsoever. Fractures spaced approximately 1m apart with tight bonding. Suitable for yielding hard, high-quality stone.</td><td rowspan="2">Elastic wave velocity
3000m/s or higher</td></tr>
<tr><td>II</td><td>The hardest rock types rich in carbonate minerals, such as limestone or breccia. Fresh, unweathered condition. Few fractures with tight bonding.</td></tr>
</table>

Table 5.2.4 shows the guidelines for classifying hard rock and soft rock in Japan. Because the classification criteria for soft rock and weathered rock in Table 5.2.1 are ambiguous, they must be categorized based on engineering judgment. For preliminary designs, the minimum ultimate bond strength value for weathered rock (0.6 MPa) is often adopted to ensure a conservative design.

Table 7.2.4 Guidelines for Ground Classification Based on Earthwork Workability using Heavy Machinery and Seismic Wave Velocity (Compiled from JRA(1986) and JRA(2009))

Ground Classification	Criteria based on construction feasibility with heavy machinery, JRA(1986)	Ground elastic wave velocity, JRA(2009a)
Soil	Excavation possible using a 21-ton class bulldozer's dozer blade	Approximately 700 m/s
Soft Rock	Excavation using a 21-ton class bulldozer's dozer blade is difficult Ripper work possible using a 32-ton class bulldozer	Approximately 2.0 km/s or less
Hard Rock	Ripper work using a 32-ton class bulldozer is difficult	Approximately 2.0 km/s or more

Tables 7.2.5 to 7.2.7 show examples of ultimate bond strength and factor of safety f_s configurations in various countries. The rock mass classifications explicitly include descriptions regarding the degree of weathering. In most of these cases, the design bond strength is determined based on the ultimate bond strength measured during testing. Furthermore, some countries vary the factor of safety depending on the specific rock type. These data clearly indicate that the ultimate bond strength of anchors varies significantly by rock type, and that a simple classification into hard rock, soft rock, and weathered rock is inadequate to capture these critical differences. Particularly for argillaceous (muddy) rocks, the ultimate bond strength values tend to be smaller compared to the Japanese Geotechnical Society (JGS) standard (Table 7.2.1). Although ground evaluation conditions vary by country, the data in Tables 7.2.5 to 7.2.7 suggest that the baseline values for the ultimate bond strength of anchors in argillaceous rocks should be estimated lower than the JGS standard (Yamada, 2009).

Table 7.2.5 International Cases of Ultimate Bond Strength and Factor of Safety for Anchors – Part 1 (Yamada, 2009, Originally published in Littlejohn & Bruce (1977))

Rock type	Working Bond (MN/m^2)	Ultimate Bond (MN/m^2)	Safety factor	Surce
Igneous				
Medium hard basalt		5.73	3.0 – 4.0	India, Rao (1964)
Weathered granite		1.50 – 2.50		Japan, Suzuki et al.(1972)
Basalt	1.21 – 1.38	3.86	2.8 – 3.2	Britain, Wycliffe-Jones (1974)
Granite	1.38 – 1.55	4.83	3.1 – 3.5	Britain, Wycliffe-Jones (1974)
Serpentine	0.45 – 0.59	1.55	2.6 – 3.5	Britain, Wycliffe-Jones (1974)
Granite & basalt		1.72 – 3.1	1.5 – 2.5	USA, PCI (1974)
Metamorphic				
Manhattan schist	0.70	2.8	4.0	USA, White (1973)
Slate & hard shale		0.83 – 1.38	1.5 – 2.5	USA,, PCI (1974)
Calcareous sediments				
Limestone	1.00	2.83	2.8	Switzerland, Losinger (1966)
Chalk (Grades I–III)	0.01 N (N=SPT in blows /0.3m	0.22 – 1.07	1.5 – 2.0 (Temporary) 3.2 – 4.0 (Permanent)	Britain, Littlejohn (1970)
Tertiary limestone	0.83 – 0.97	2.76	2.9 – 3.3	Britain, Wycliffe-Jones (1974)
Chalk limestone	0.86 – 1.00	2.76	2.8 – 3.2	Britain, Wycliffe-Jones (1974)
Soft limestone		1.03 – 1.52	1.5 – 2.5	USA, PCI (1974)
Dolomitic limestone		1.38 – 2.07	1.5 – 2.5	USA, PCI (1974)
Arenaceous sediments				
Hard coarse-grained sandstone	2.45		1.75	Canada, Coates (1970)
Weathered sandstone		0.69 – 0.85	3.0	New Zeeland, Irwin (1971)
Well-cemented mudstone		.0.69	2.0 – 2.5	New Zeeland, Irwin (1971)
Bunter sandstone*1	0.40		3.0	Britain, Littlejohn (1970)
Bunter sandstone (UCS>2 Mn/m^2)	0.60		3.0	Britain, Littlejohn (1970)
Hard fine sandstone	0.69 – 0.83	2.24	2.7 – 3.3	Britain, Wycliffe-Jones (1974)
Sandstone		0.83 – 1.73	1.5 – 2.5	USA, PCI (1974)
Argillaceous sediments				
Keuper marl*2		0.17 – .025 ($0.45\ C_u$)	3.0	Britain, Littlejohn (1970) (C_u = undrained cohesion)
Weak shale		0.35		Canada, Golder Brawner (1973)
Soft Sandstone & Shale	0.10 – 0.14	0.37	2.7 – 3.7	Britain, Wycliffe-Jones (1974)
Soft Shale		0.21 – 0.83	1.5 – 2.5	USA, PCI (1974)
Weak rock	0.35 – 0.70			
Medium rock	0.70 – 1.05			Australia, Koch (1972)
Strong rock	1.05 – 1.40			
Wide variety of igneous and metamorphic rocks	1.05		2.0	Australia, CA35 (1972)
Wide variety of rocks	0.98			France, Fargeot (1972)
	0.50			Switzerland, Walther (1969)
	0.70			Switzerland, Comte (1965)
		1.20 – 2.50		Switzerland, Comte (1971)
	0.70		2.0 – 2.5 (Temporary) 3.0 (Permanent)	Italy, Mascardi (1973)
	0.69	2.76	4.0	Canada, Golder Brawner (1973)
	1.40	4.20	3.0	USA, White (1973)
Concrete		1.38 – 2.76	1.5 – 2.5	USA, PCI (1974)

*1: In Europe, the term “Bunter” is used to refer to the Lower Triassic stratigraphic unit of the Mesozoic Era.
*2: “Keuper” is the term used in Europe to refer to the Upper Triassic stratigraphic unit of the Mesozoic Era.

Table 7.2.6 International Cases of Ultimate Bond Strength and Factor of Safety for Anchors – Part 2 (Yamada, 2009, Originally published in Littlejohn & Bruce (1977))

Rock type	Working bond (MN/m^2)	Test bond (MN/m^2)	Ultimate bond (MN/m^2)	Safety factor (test) Sm	Safety factor (ultimate) Sf	Source
Igneous						
Basalt	1.93		6.37		3.3	Britain, Paker (1958)
Basalt	1.10	3.60		3.27		USA, Eberhard & Veltop (1955)
Tuff	0.80					France, Camberfort(1966)
Basalt	0.63	0.72		1.14		Britain, Cementation(1962)
Granite	1.56	1.72		1.1		Britain, Cementation(1962)
Dolerite	1.56	1.72		1.1		Britain, Cementation(1962)
Very fissured felsite	1.56	1.72		1.1		Britain, Cementation(1962)
Very hard dolerite	1.56	1.72		1.1		Britain, Cementation(1962)
Hard granite	1.56	1.72		1.1		Britain, Cementation(1962)
Basalt and tuff	1.56	1.72		1.1		Britain, Cementation(1962)
Granodiorite	1.09					Britain, Cementation(1962)
Shattered basalt		1.01				USA, Saliman & Schaefer(1968)
Decomposed granite		1.24				USA, Saliman & Schaefer(1968)
Flow breccia		0.93				USA, Saliman & Schaefer(1968)
Mylonitised porphyrite	0.32 – 0.57					Switzerland, Descoeudres(1969)
Fractured diorite	0.95					Switzerland, Descoeudres(1969)
Granite	0.63	0.81		1.28		Canada, Barron et al (1971)
Metamorphic						
Schist	0.31					Switzerland, Birkenmajer (1953)
All types	1.20					Finland, Maijala (1966)
Weathered fractured quartzite	1.56	1.72		1.1		Britain, Cementation(1962)
Blue schist	1.52	1.67		1.1		Britain, Cementation(1962)
Weak meta sediments	1.10	1.23		1.1		Britain, Cementation(1962)
Slate	0.43					Britain, Cementation(1962)
Slate/meta greywacke	1.57	1.73		1.1		Britain, Cementation(1962)
Granite gneiss	0.36 – 0.69					Sweden, Broms (1968)
Folded quartzite	0.51					Australia, Rawlings (1968)
Weathered meta tuff		0.29				USA, Saliman & Schaefer(1968)
Graywacke	0.34					Germany, Heifold & Schaefer (1969)
Quartzite	0.93 – 1.20	1.02 – 1.32		1.1		Britain, Gossch alk & Taylor (1969)
Microgneiss	0.95					Italy, Mantovani (1970)
Metamorphic						
Sericite schist	0.05					Italy, Berardi (1972)
Quartzite/schist	0.10					Italy, Berardi (1972)
Argillaceous & calcareous Schist	0.63					Italy, Berardi (1972)
Slate	0.95	1.24		1.3		Switzerland, Moschler & Matt
Highly metasediments	0.83	1.08		1.3		USA, Buro (1972)
Slate & greywacke	1.08	1.40		1.3		Germany, Anom (1972)
Various metasediments			1.57			Germany, Abraham & Porzig (1973)
Micaschist/biotite gneiss	0.53	0.80		1.5		USA, Nicholson Anch Co. Ltd. (1973)
Slate	0.60	0.90	1.80	1.5	3.0	Britain, LittleJohn et al.(1974)
Sound Micaschist	1.74	2.16		1.24		USA, Feld & White (1974)
Micaschist	0.52 – 0.74					USA, Feld & White (1974)
Very-poor gneiss & mud band	0.07					USA, Feld & White (1974)
Carbonate sediments						
Loamy limestone		0.63				Italy, Berardi (1972)
Fissured limestone & intercalations	1.08	1.19		1.1		Britain, Cementation Co. Ltd. (1962)
Limestone	0.65					Switzerland, Muller (1966)
Poor limestone	0.32					France, Hennequin et al. (1966)
Massive limestone	0.39 – 0.78					France, Hennequin et al. (1966)
Karstic limestone	0.54					France, Hennequin et al. (1966)
Tertiary Limestone	1.00		2.83		2.8	Switzerland, Losinger Co. Ltd.(1966)
Limestone			4.55 – 4.80			Switzerland, Rutter (1966)
Marly limestone	0.03 – 0.07	0.21 – 0.36				Italy, Berardi (1967)
Limestone			0.27			USA, Saliman et al.(1968)
Limestone	0.28					Italy, Berardi (1969)
Dolomitic Limestone			1.80			Canada, Brown (1970)
Marly limestone	0.39 – 0.94					Italy, Berardi (1972)
Limestone	0.26					Italy, Berardi (1972)
Limestone/ puddingstone	0.44					France, Soletanche Co. Ltd.(1968)
Limestone	1.18	1.42		1.2		USA, Buro (1972)
Chalk			0.07			Britain, Associated Tum Co. Ltd.(1973)
Dolomite		1.66				Canada, Golder Brawner (1973)
Dolomitic siltstone	0.43					USA, White (1973)
Limestone & marly bands	.0.37	0.55		1.5		Italy, Mongilardi (1972)

Table 7.2.7 International Cases of Ultimate Bond Strength and Factor of Safety for Anchors – Part 3 (Yamada, 2009, originally published in Littlejohn and Bruce (1977))

Rock type	Working bond (MN/m^2)	Test bond (MN/m^2)	Ultimate bond (MN/m^2)	Safety factor (test) Sm	Safety factor (ultimate) Sf	Source
Arenaceous sediments						
Sandstone	1.44	1.58		1.1		Britain, Morris et al (1956)
Hard Sandstone	1.42	1.56		1.1		Britain, Cementation Co. Ltd. (1962)
Bunter Sandstone*1	0.95	0.98		1.03		Britain, Cementation Co. Ltd. (1962)
Sandstone	0.76	0.84		1.1		Britain, Cementation Co. Ltd. (1962)
Sandstone	0.74					Czechoslovakia, Hobst (1968)
Sandstone	0.31	0.40	1.73	1.3	5.6	UAS, Drossel (1970
Sandstone	0.80					USA, Thompson (1970)
Poor Sandstone	0.40					Germany, Brunner (1970)
Good Sandstone	1.14					Germany, Brunner (1970)
Sandstone & Breccia	0.38					France, Soletanche (1968)
Sandstone		0.95				Australis, Williaws et al.(1972)
Bunter Sandstone*1	0.60	1.20		2.0		Britain, Littlejohn (1973)
Sandstone	1.17					Australis, Mcleod (1974)
Argillaceous sediments						
Shale	0.62					Canada, Jueraens (1965)
Marl	0.10	0.28		2.8		Italy, Berardi (1967)
Shale	0.30		0.63		2.1	Canada, Hanna et al.(1967)
Very weathered shale			0.39			USA, Saliman et al.(1968)
Shale	0.13 – 0.24					USA, Koziakin (1970)
Siltstone	0.62					Britain, Uni Anth Co. Ltd.(1972)
Clay marl	0.14 – 0.24	0.21 – 0.36				Germany, Schwarz (1972)
Shale	0.62			1.5		Canada, McRosite (1973)
Argillite	0.82					Canada, Golder Brawner (1973)
Mudstone	0.63	0.88		1.4		Australis, Mcleod (1974)
Miscellaneus						
Bedded sandstone & shale	0.25 – 0.50					Italy, Beomonte(1961)
Porous, sound goassamer	1.57	1.72		1.1		Britain, Cementation Co. Ltd. (1962)
Shale & sandstone	0.07	0.10		1.5		USA, Reti(1964)
Soft rocks	0.75					Sweden, Nordin(1966)
Sandstone & shale	1.82					Poland, Bujak et al.(1967)
Siltstone & mudstone	1.65					Australia, Maddox et al.(1967)
Fractured rock						
(75 per cent shale)						
Poor			0.24			USA, Saliman & Schaefer (1968)
Average			0.35			USA, Saliman & Schaefer (1968)
Good			0.75			USA, Saliman & Schaefer (1968)
Limestone & caly breccia	0.20 – 0.23					Italy, Berardi (1972)

*1: In Europe, the term “Bunter” is used to refer to the Lower Triassic stratigraphic unit of the Mesozoic Era.

(b) Factor of Safety for Ultimate Bond Strength

The factors of safety for ultimate bond strength adopted in Japan are shown in Table 7.2.8. These values differ significantly from the international cases shown in Tables 7.2.5 to 7.2.7. Table 7.2.9 details the factor of safety values for anchors in various countries prior to 1979. After referencing these international practices, the factor of safety for non-seismic conditions listed in Table 7.2.8 was proposed by a committee on anchor design established within the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE), the predecessor of the Japanese Geotechnical Society (JGS). Because Japan features highly heterogeneous ground conditions shaped by frequent slope movements, seismic disasters, and active orogenic processes, it is understood that a slightly larger factor of safety was intentionally set compared to other nations.

Table 7.2.8 Factors of Safety f_s for Ultimate Pull-out Capacity Adopted in Japan (JGS, 2012)

Anchor Classification		Safety Factor f_s
Rank B (Temporary Use)		1.5
Rank A	Non-seismic conditions	2.5
	Seismic conditions	1.5～2.0

Table 7.2.9 International Standard Baseline Values for Anchor Factors of Safety around 1976 (JSSMFE, 1976) (Table title in source: List of Proposed Safety Factors (7th International Conference of S.M.F.E))

Proposer	Safety Factor	Notes	
Draft Earth Anchor Regulations of West Germany	1.5	When design load is calculated using active earth pressure	Safety factor applies to the maximum load actually applied to the anchor
	1.33	When design load is calculated using passive earth pressure	
Descoeuders (Switzerland)	1.5	For the maximum load actually applied to the anchor, including both temporary and permanent anchors	
Maeste (France)	2.0	Permanent Anchor	
	1.5	Temporary Anchor	
Jorge (France)	1.5	Load at which creep begins	
Criveli (Switzerland)	1.5	Temporary Anchor calculated based on Coulomb earth pressure	

Note: SMFE: Soil Mechanics & Foundation Engineering

(c) Allowable Bond Stress Between Tendons and Grout

Earth Anchors (JSSMFE, 1976), published in 1976, noted that empirical data on the allowable bond stress between PC strands and grout were scarce, and thus introduced the bond strength values for reinforcing bars from the Architectural Institute of Japan (AIJ) at that time as temporary reference values. The 1990 edition of *Design and Construction Standards for Ground Anchors and Commentary* (JSSMFE, 1990) presented its own original baseline values while explaining that they were referenced from the Standard Specifications for Concrete Structures by the Japan Society of Civil Engineers (JSCE). Although the notation was converted to SI units by the Japanese Geotechnical Society (JGS, 2000) in 2000, the absolute values themselves were inherited without change. In the 2012 edition of *Design and Construction Standards for Ground Anchors and Commentary* (JGS, 2012), the anchor classification system was thoroughly revised, establishing the standard values currently used today, as shown in Table 7.2.10.

Table 7.2.10: Allowable Bond Stress Between Grout and Tendons (MN/m²) (JGS, 2012)

Classification	Design Strength of Grout / Type of Tensile Material	18	24	30	40 or more
Grade B	PC Steel Wire PC Steel Bar PC Steel Strand Multiple PC Steel Strand	1.0	1.2	1.35	1.5
	Deformed PC Steel Bar	1.4	1.6	1.8	2.0
Grade A	PC Steel Wire PC Steel Bar PC Steel Strand Multiple PC Steel Strand	—	0.8	0.9	1.0
	Deformed PC Steel Bar		1.6	1.8	2.0

(d) Minimum Limit for Free Length

Table 7.2.11 presents examples of anchor free lengths from international standards introduced in *Design and Construction Standards for Ground Anchors and Commentary* (JSSMFE, 1990). Referencing these international practices and considering operational factors such as the ease of tension control, a committee within the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE) in 1990 established a unique Japanese standard requiring a minimum free length of 4 m or more in principle. When the free length falls below 4 m, the following technical issues arise (JGS, 2012):

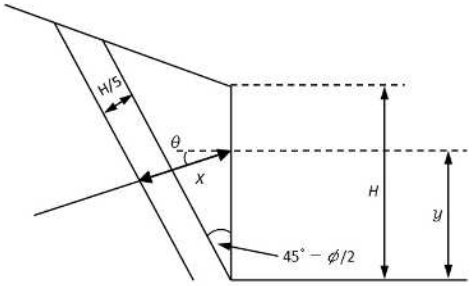
i) The ground between the reaction structure and the fixed anchor (bond) length may fail.

ii) In the wedge anchorage system, due to the anchor set (draw-in) during stressing and anchoring, a shorter free length requires a larger initial jacking force to be introduced into the tendon, which may exceed the specified maximum allowable load.

iii) In the wedge anchorage system, it also becomes highly difficult to precisely control and set the initial tension force to the design value.

Table 7.2.11 International Standard Baseline Values for Anchor Free Length Introduced by JSSMFE (1990)

Overseas Standard	Tendon Free Length	
U.S. Dapartment of Transportation Federal Highway Administration	Rock, Soil	Extend the slip surface by at least 5 ft (1.5 m).
	Retaining Wall	Length equal to the slip surface position plus 1/5 of the maximum retaining wall height (see Explanatory Diagram 7.1)
	(Concept) 1) The soil and tendon sections must not be compressed due to prestressing. 2) The area near the slip line must have a sufficient safety factor and not become a passive zone.	



$$\text{Minimum free length (X)} = \frac{y \cdot \sin\left(45^\circ - \frac{\phi}{2}\right)}{\sin\left(45^\circ + \theta + \frac{\phi}{2}\right)} + \frac{H}{5\sin\left(45^\circ + \theta + \frac{\phi}{2}\right)}$$

Explanation Diagram 7.1 Example of Evaluating the Minimum Free Length
(U.S. Dapartment of Transportation Federal Highway Administration: FHWA)

(e) Lower and Upper Limits for Fixed Anchor Length

Table 7.2.12 presents examples of regulations for fixed anchor lengths in international standards introduced in *Design and Construction Standards for Ground Anchors and Commentary* (JSSMFE, 1990). Referencing these international practices, a committee within the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE) in 1990 established a unique Japanese standard requiring a fixed anchor length of between 3 m and 10 m in principle.

Table 7.2.12 International Standard Baseline Values for Fixed Anchor Length Introduced by JSSMFE(1990)

Overseas Standards	Fixed Anchor Length
1. BSI (DD81) FLP	Fixed Anchor Length——3m or more, 10m or less 1) The embedment length is determined by the excavation method, soil conditions, load, and grout pressure. 2) Embedment length is longer for cohesive soils than for sandy soils.
2. PTI	Fixed Anchor length——4.6m or more 1) The failure bond stress in rock is determined by the rock shear strength, ground characteristics, drilling method, borehole diameter, and grout strength. 2) The failure bond stress in soil is determined by the drilling method, soil conditions, consolidation conditions, borehole diameter, and grout strength. 3) Cohesive soils allow for a longer fixed anchor length than sandy soils.

Although it is possible to shorten the fixed anchor length when the design anchor force is small, an extremely short length may cause the ultimate pull-out capacity to vary significantly due to minor variations in the inclination or thickness of the geological strata within the anchorage ground. To minimize such adverse effects, the minimum fixed anchor length is set to 3 m. Figure 7.2.2 illustrates the relationship between the ultimate pull-out capacity of bonded pre-stressed anchors and the fixed anchor length. When the design anchor force is large, the ultimate pull-out capacity per anchor does not increase proportionally even if the fixed anchor length is extended, and it virtually plateaus beyond 10 m. For these reasons, the fixed anchor length is restricted to 10 m or less (JGS, 2012).

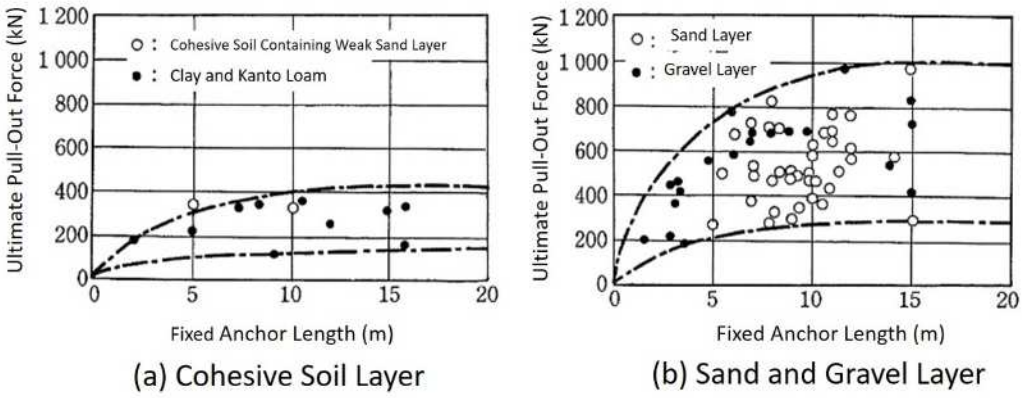


Figure 7.2.2 Relationship Between Fixed Anchor Length and Ultimate Pull-out Capacity (JGS, 2012)

(3) Actions to Take

(a) Ultimate Bond Strength

Because geological characteristics overseas differ in both geological age and rock types depending on the nation, ground classification systems using parameters such as geology, geological age, weathered/unweathered conditions, and soft/hard rock should fundamentally align with the local geological conditions of the target country. If no issues exist with the corresponding existing standard baseline values for ultimate bond strength, those local values should be adopted. In cases where such baseline values do not exist, or where engineering problems have occurred with existing local standards, Japan's standard baseline values should be referenced as a supplemental guide.

On the premise that ultimate bond strength will ultimately be verified through anchor investigation tests, it is desirable to establish the standard baseline values for preliminary design based on Japanese standards while referencing the aforementioned international case studies. Japan's ground classification system (hard rock, soft rock, and weathered rock), which is defined by earthwork workability (excavatability) using heavy machinery, is universally applicable in other nations. Although the criteria for distinguishing between soft rock and weathered rock are ambiguous, it is practical to categorize "Soft Rock I" as weathered rock in engineering practice.

(b) Factor of Safety f_s for Ultimate bond strength

Similar to the ultimate bond strength described above, if the target country has its own factors of safety tailored to its local geological conditions and no issues exist with them, those values should be utilized. Although the factor of safety f_s against ultimate pull-out capacity adopted in Japan references the values set in Europe and North America around 1979, a slightly larger value was intentionally adopted in consideration of Japan's highly heterogeneous ground conditions. In cases where engineering problems exist with the target country's existing factors of safety, it is desirable to establish the values based on Japan's standard baseline values while referencing international case studies.

(c) Allowable Bond Stress Between Tendons and Grout

Because the specifications for PC wires and strands may differ depending on the nation, it is desirable to adopt the target country's reference values for the specific PC materials being used. If such standards do not exist, Japan's standard baseline values (Table 7.2.10) should be utilized as a base.

(d) Minimum Limit for Free Length

Because the minimum free length (4 m) adopted in Japan was established based on values used in Europe and North America around 1990 as well as extensive experience in tension control, it serves as a highly universal value. Therefore, it is desirable for other nations to also adopt Japan's limit value as their standard.

(e) Lower and Upper Limits for Fixed Anchor Length

Because the fixed anchor length adopted in Japan (3 to 10 m, though 7 m or less is preferable) was established based on values used in Europe and North America around 1990 as well as extensive experience in tension control, it serves as a highly universal value. Therefore, it is desirable for other nations to also adopt Japan's limit values as their standards.

References

JSSMFE (1976): *Earth Anchor*s, The Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE), 236p.

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7. 2. 3 Lack of Uniformity in the Design Methods for Cast-in-place Reaction Structures

(1) Issues

Although cast-in-place reaction structures are frequently selected based on economic comparisons, the design methodologies for these reaction plates in Japan are not standardized.

(2) Explanation

Design methodologies for factory-made reaction structures, accommodating various structures and materials, are detailed in the *Ground Anchor Reaction Structure Design and Testing Manual* (PWRC, 2004), which standardizes the design approach for each type of reaction structure. Reinforced concrete (RC) reaction structures share identical structures and functions regardless of whether they are factory-made or cast-in-place; the only differences lie in concrete strength and the tolerance of structural details. Therefore, it is desirable to standardize the design methods for all RC reaction structures based on the methodology outlined in this manual.

(3) Actions to Take

The design methodologies for cast-in-place reaction plates shall be standardized based on the reinforced concrete (RC) reaction structure design method specified in the *Ground Anchor Reaction Structure Design and Testing Manual* (PWRC, 2004).

References

PWRC (2004): *Ground Anchor Reaction Structure Design and Testing Manual*, Public Works Research Center (PWRC).

7. 2. 4 Lack of Infrastructure for Various Field Tests to Obtain Design Parameters

(1) Issues

Although various field tests, such as standard penetration tests and anchor investigation tests, are routinely conducted in Japan, and multiple technical documents regarding these testing methodologies exist, such field tests are generally not performed in developing countries.

(2) Explanation

If technical documents regarding field testing are not established in the target country, it is necessary to incorporate detailed explanations of these testing methodologies directly into the *Ground Anchor Design Manual*.

Earth Anchors (JSSMFE, 1976)—Japan's first book on the design and construction of ground anchors, published by the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE), the predecessor of the Japanese Geotechnical Society (JGS)—describes the methodologies for investigation, suitability, and acceptance tests. These testing methodologies were established by referencing the details of the investigation, suitability, and acceptance tests specified in the standard for temporary anchors (*DIN 4125-1972*) issued by the German Institute for Standardization (DIN). *Earth Anchors* (JSSMFE, 1976) includes a complete Japanese translation of *DIN 4125-1972* as reference material. The current latest DIN standard for anchors is available as *DIN 4125-1990* (*Ground Anchorages; Design, Construction and Testing*), which explicitly details the procedures for these investigation, suitability, and acceptance tests.

Furthermore, the standard penetration test (JIS A 1219), as well as the anchor investigation, suitability, and acceptance tests conducted in Japan, were originally standardized in Europe and North America and subsequently adopted in Japan. International standards equivalent to the standard penetration test include ASTM D1586 (USA), BS EN ISO 22476-3 (UK), and AS 1289.6.3.1 (Australia).

(3) Actions to Take

Because various anchor-related tests and standard penetration tests have become international standards, it is highly desirable for developing countries to procure and utilize them. Standard penetration tests can sometimes be successfully conducted by local drilling contractors even in developing countries. For nations where standard penetration tests and various anchor-related tests are not currently performed, it is necessary to carry out technology transfer to local engineers. For the sake of user convenience, this document describes only the methodologies for anchor investigation tests in a later section.

References

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7. 3 Design Considerations

7. 3. 1 Conditions for Applying Shear-resistance Increasing Effect

The effects of ground anchors are divided into the shear-resistance increasing effect and the driving force reduction effect. The shear-resistance increasing effect occurs when the anchor force increases the effective stress on the slip surface, thereby increasing its shear resistance; however, this effect becomes relatively small when the slip surface inclination is gentle. Furthermore, in slow-moving landslides where the moving layer thickness reaches several tens of meters, there is an engineering view that the weathered rock within the moving layer creates an arching effect that supports the overburden load like a bridge, meaning that 100% of the normal component of the anchor force applied at the ground surface never reaches the slip surface. For steep slope failures or collapse-type landslides, the slip surface inclination is steep, making the shear-resistance increasing effect relatively large, and the moving layer thickness is also relatively shallow; therefore, an approach that considers both the driving force reduction and shear-resistance increasing effects is highly economical and effective. Conversely, in typical slow-moving landslides in a narrow sense where the moving layer thickness is often large (such as reactivated slow-moving landslides), the slip surface inclination is gentle, resulting in a relatively small shear-resistance increasing effect; thus, a method that considers only the driving force reduction effect is a safer approach.

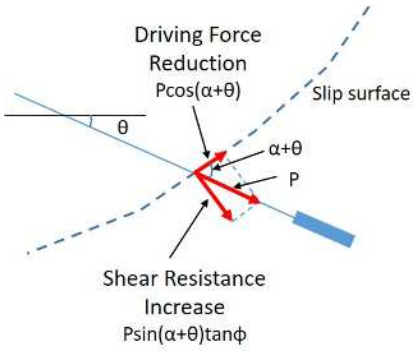


Figure 7.3.1 Shear-resistance Increasing Effect and Driving Force Reduction Effect of Ground Anchors

7. 3. 2 Restrictions on the Inclination Angle of Anchors

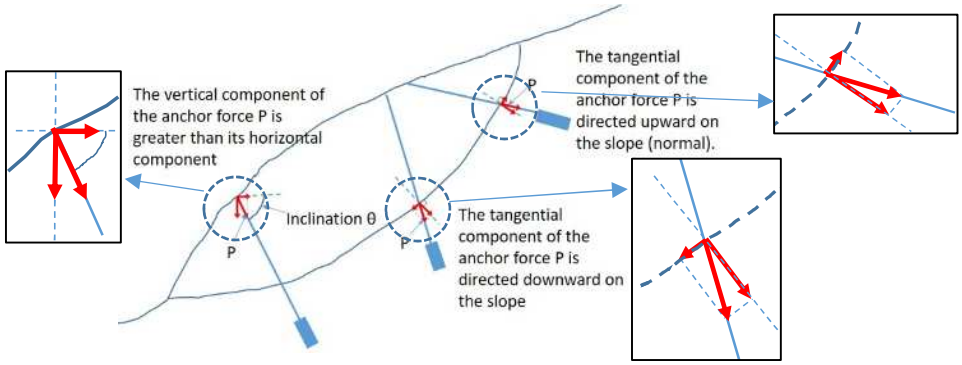


Figure 7.3.2 Examples of Anchor Inclination Angles that Actively Contribute to Slope Destabilization

For anchors intended for slope and cut-slope stabilization, the anchor inclination angle is designed to be 45° or less, ensuring that the tangential component of the anchor force points in the upslope direction (see the top diagram of Figure 7.3.2). If the tangential component of the anchor force points in the downslope direction due to the intersection angle between the anchor axis and the slip surface, as shown in the middle diagram of Figure 7.3.2,

there is a distinct risk of destabilizing the slope. Similarly, designing with an inclination angle greater than 45°, as shown in the bottom diagram of Figure 7.3.2, causes the vertical component of the anchor force to become larger than its horizontal component, posing a critical risk that the anchor force will actively contribute to the destabilization of the slope or cut slope. Furthermore, if the anchor inclination angle is within the range of -5° to +5°, a sound fixed anchor (bond) length cannot be formed due to residual slime and grout bleeding generated during construction, which may significantly impair the load-bearing capacity of the anchor; therefore, this specific range must be avoided (JAA, 2013).

7. 3. 3 Restrictions on the Free Length of Anchors

The standard free length for ground anchors is 4 m or more. The technical issues that arise when the free length falls below 4 m are detailed in Section 7.2.2.

7. 3. 4 Restrictions on the Fixed Anchor Length

As a rule, the standard fixed anchor length is set between 3 m and 10 m in principle. Although it is possible to shorten the fixed anchor length when the design anchor force is small, an extremely short length may cause the ultimate pull-out capacity to vary significantly due to minor variations in the inclination or thickness of the geological strata within the anchorage ground. To minimize such adverse effects, the minimum fixed anchor length is set to 3 m. Figure 7.3.2 illustrates the relationship between the ultimate pull-out capacity of friction-type anchors and the fixed anchor length. When the design anchor force is large, the ultimate pull-out capacity per anchor does not increase proportionally even if the fixed anchor length is extended, and it virtually plateaus beyond 10 m. For these reasons, the fixed anchor length is restricted to 10 m or less (JGS, 2012).

7. 3. 5 Setting the Lock-off Tension Force

In typical reactivated slow-moving landslides, since the inclination angle of the slip surface is relatively gentle, anchors are often designed to rely solely on the driving force reduction effect. In such cases, it is highly recommended to set the lock-off tension force to approximately the design anchor force (around 80% to 90%). In Japan, this lock-off tension force is sometimes set to around 50% of the design anchor force; however, if the lock-off tension is this small, the slow-moving landslide can easily slide, causing the anchor tendon to undergo bending deformation at the slip surface position, which leads to the failure of components surrounding the tendon. If such deformation or failure occurs, the anchor force applied at the ground surface will not be sufficiently transmitted to the stable stratum, resulting in the critical adverse effect of reduced restraining performance.

The inherent characteristics of ground anchors that must be considered when determining the lock-off tension force are presented below:

1) Displacement of reaction structures and subsequent changes in the residual tensile force due to ground creep
2) Reduction in residual tensile force due to tendon relaxation
3) Attenuation of the transmitted load due to friction between the tendon and sheath within the free length
4) Changes in residual tensile force due to creep of the anchor pedestal and the supported structure
5) Changes in residual tensile force due to time-dependent changes in the restraining force within the bond zone and the deformation or creep of the grout 6) Fluctuations in the residual tensile force due to changes in external forces accompanying long-term situational variations after anchoring

7. 3. 6 Reduction in Design Anchor Force Due to Friction Loss in the Free Length

Ground anchors for slope stabilization must reliably transmit the anchor force to the fixed anchor (bond) length. The actual force transmitted to the bond zone is the value obtained by subtracting the amount of friction loss in the free length from the anchor force applied at the ground surface, and this effective value serves as the restraint force. The Japanese Geotechnical Society (JGS, 2012) explains that the amount of friction loss R_v in the free length is determined from the results of anchor investigation tests, as shown in Figure 7.3.3.

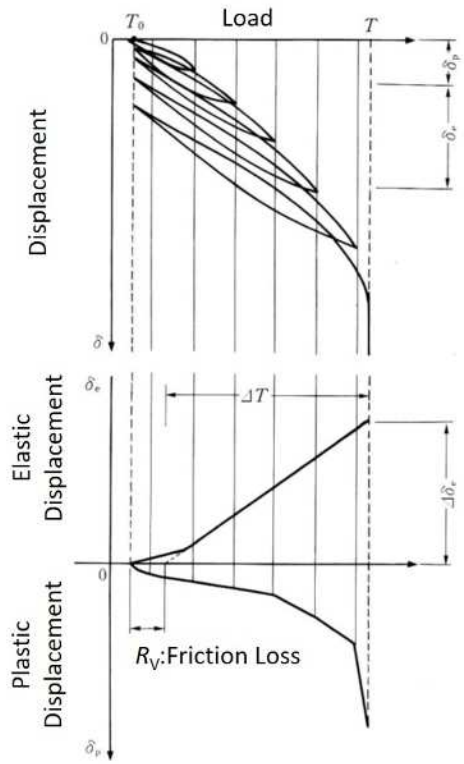
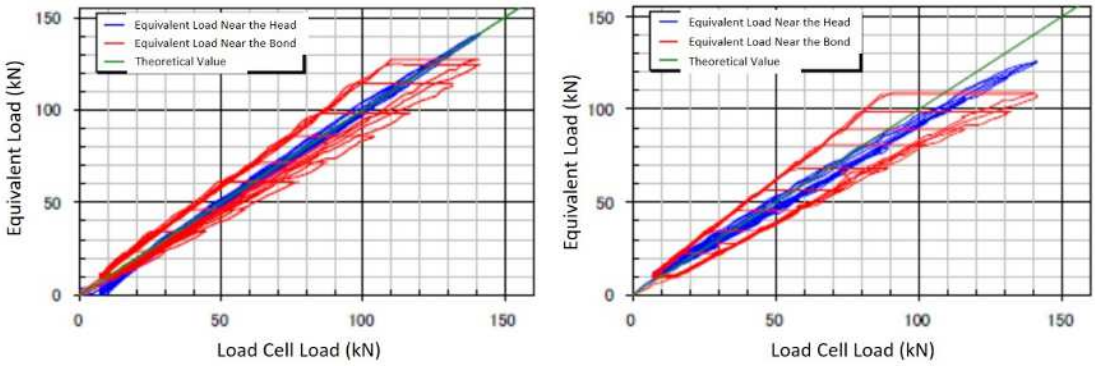


Figure 7.3.3 Methodlogy for Determining Friction Losses within the Free Length (JGS, 2012)

Takyu et al. (2010) investigated the relationship between the applied load at the ground surface and the load transmitted near the bond zone using actual ground anchors. Figure 7.3.4 shows an example of their findings, presenting the measurement results for cases with a free length L_f of 30 m and 50 m. The horizontal axis represents the applied load at the ground surface, while the vertical axis shows the load transmitted near the bond zone. Observing the trend indicated by the red lines, approximately 10% of the applied load is converted into friction loss for the 30 m free length, and approximately 20% of the applied load becomes friction loss for the 50 m free length. This reveals the following points:

1) As the free length increases, friction losses increase.
2) Friction losses are approximately proportional to the applied load.

The fact that the amount of friction loss is proportional to the applied load cannot be explained by the Japanese Geotechnical Society (JGS) method shown in Figure 7.3.3. This implies that the amount of friction loss is already included within the slope of $\Delta\delta_e/\Delta T$ in the elastic displacement portion of Figure 7.3.3. Consequently, it is necessary to treat the net value obtained by excluding this friction loss as the true restraint force.



(a) L_f = 30m **(b) L_f = 50 m**

Figure 7.3.4 Relationship Between Applied Load at the Ground Surface and the Load Transmitted Near the Bond Zone (Takyu et al., 2010)

Based on these experimental results, Takyu et al. (2010) proposed the following formula to calculate the tensile load efficiency factor (β), which represents the rate at which the tensile load applied at the ground surface reaches the depth of the fixed anchor length, accounting for the friction loss in the free length.

$$\beta = 1 + 94.6L_f(1 - L_f) \times 10^{-6}$$

Here, β : Tensile load efficiency factor

L_f : Free length (m)

This calculation formula incorporates the friction loss ($\boldsymbol{R_v}$) described in Figure 7.3.4. Because the impact of friction loss within the free length section becomes increasingly significant as the free length extends, it is desirable to calculate the actual load transmitted to the fixed anchor length using this formula as a reference, thereby determining a design anchor force that explicitly accounts for these losses.

References

JGS (2012): *Design and Construction Standards for Ground Anchors, and Explanatory Notes*, The Japanese Geotechnical Society (JGS), 211P.

JAA (2013): *Ground anchor design/ construction instruction manual*, Japan Anchor Association (JAA), 263P.

Takyu, T., Shimoda, K., Kawasaki, H., Tamura, T. (2010): Friction loss in free length of Ground anchor, *Japanese Geotechnical Journal of the Jpn. Geotechnical Soc*., Vol.5, No.2, pp.281-290.

7. 3. 7 Anchor Investigation Tests

Although the details of the tests used to determine various design parameters for ground anchors are extensively described in relevant technical documents in Japan, an explanation of this testing methodology is also essential in target countries where the development of such technical documents is still limited.

International standards, such as DIN 4125-1990, exist for anchor investigation, suitability, and acceptance tests, and Japanese standards are compliant with these regulations. Investigation tests are divided into pull-out tests and long-term tests. Pull-out tests are conducted to understand the ultimate pull-out capacity of the anchor and its behavior, as well as to determine various design parameters used in the anchor design. Since the various parameters of the anchorage ground can vary significantly depending on the region or specific location, it is desirable to

confirm them through field pull-out tests. The configurations of the test anchors, planned maximum loads, and other factors differ depending on whether the objective is to determine the ultimate pull-out capacity, the ultimate bond strength, or the ultimate bearing resistance.

(1) Test Anchor

Because the ultimate pull-out capacity of an anchor can vary significantly depending on the anchor type and installation methodology, the test anchors used for pull-out testing must be installed using the identical method as the permanent anchors intended for actual service. When determining the ultimate bond strength or ultimate bearing resistance, it is necessary to employ an installation layout that prevents the friction resistance of the ground above the fixed anchor (bond) length from acting. As a practical example, a bag packer is attached to the top of the bond zone to take appropriate measures that eliminate pull-out resistance within the free length.

(2) Design Maximum Test Load

The loading stages up to the planned maximum test load must be configured so that the test anchor reaches its ultimate state-meaning that an ultimate failure occurs between the ground and the fixed anchor (bond) length, leading to pullout failure. Because the methodology for determining the planned maximum test load differs depending on whether the objective is to determine the ultimate pull-out capacity, the ultimate bond strength, or the ultimate bearing resistance, the criteria must be selected accordingly.

In the former case, the planned maximum test load is set to either the load obtained by multiplying the design anchor force by a prescribed factor of safety, or a load approximately 10% to 20% larger than the design anchor force. If the planned maximum test load exceeds 0.9 times the yield tensile strength or 0.7 times the guaranteed tensile strength of the tendon, appropriate countermeasures must be taken, such as making the fixed anchor length of the test anchor slightly shorter than the design fixed anchor length. In the latter case, the fixed anchor length is intentionally shortened to a few meters, or the cross-sectional area of the tendon is made as large as possible, to ensure that a complete pullout failure can be achieved.

(3) Testing Methodologies Method

Loading is generally performed using a multi-stage cyclic loading method with five or more cycles, and it must be confirmed that the load can be maintained for a specified duration during each loading stage. The guidelines for the load retention time in each cycle are presented below, which shall be appropriately adjusted according to the specific bearing characteristics of the anchorage ground.

Table 7.3.1 Load Duration

Anchorage Ground / Load Type	Cohesive Soil	Sandy Soil/Rock	Remarks
New Load Stage	Constant time of 10 min or longer	Constant time of 5 min or longer	However, if displacement does not stabilize, maintain the load until stabilization occurs.
Load Stage within History	Constant time of 2 min or longer	Constant time of 1 min or longer	

(4) Compilation and Evaluation of Test Results

The test results are compiled in the form of a load-displacement curve. Furthermore, the total displacement is divided into elastic displacement and plastic displacement, and these are plotted in the form of a load-elastic

displacement curve and a load-plastic displacement curve. The following parameters are evaluated from these compiled results:

1) Ultimate Pull-Out Capacity

The ultimate pull-out capacity is defined as the load at the point where the load-displacement curve plateaus (ceases to increase), or where the plastic displacement abruptly increases. For bonded pre-stressed anchors, the ultimate bond strength τ_u is evaluated using the following formula:

$$\tau_u = T_{ug} / \pi \cdot d_a \cdot l_s \quad \cdots (7.3.1)$$

Here, T_{ug}: Ultimate Pull-Out Force (N)

d_a: Anchor Body Diameter (mm)

l_s: Fixed Anchor Length (mm)

Furthermore, for bonded pre-stressed anchors, the ultimate pull-out capacity of an anchor with a long fixed anchor length does not increase as a simple proportional multiple of that of a shorter anchor due to the effect of progressive failure. Therefore, when estimating the magnitude of the bond strength τ to be used in the design of an anchor with a long fixed anchor length from the ultimate bond strength τ_u obtained from a short test anchor, it is necessary to apply an appropriate reduction factor.

2) Friction Loss in the free length

Friction loss R_v is calculated using the following formula.

$$R_v = (1 - \beta) \times T$$

$$\beta = 1 + 94.6 L_f (1 - L_f) \times 10^{-6}$$

Here, R_v : Friction loss (kN)

T: Anchor design load (kN)

β: Tensile load efficiency factor

L_f: Free stressing length (m)

3) Tendon Free Length

The tendon free length l_{sf} is determined from the slope of the linear portion of the load-elastic displacement curve using the following equation.

$$l_{sf} = K \cdot E_s \cdot A_s = \Delta\delta_e \cdot E_s \cdot A_s / \Delta T \quad \cdots (7.3.2)$$

Here, l_{sf} : Tendon free length (mm)

K: Slope of the linear portion of the load-elastic displacement curve (N/mm)

E_s: Elastic modulus of the tendon (N/mm²)

A_s: Cross-sectional area of the tendon (mm²)

$\Delta\delta_e$: Displacement on the linear portion of the load-elastic displacement curve (mm)

ΔT : Load on the linear portion of the load-elastic displacement curve (N)

The amount of friction loss in the free length of the anchor and the effective tendon free length value shall be utilized as key reference data for evaluating the anchor spring stiffness during permanent anchor design, as well

as for design and construction considerations of the free length section. Furthermore, for tension-type friction anchors, the effective tendon free length value serves as vital reference data for estimating the debonding failure zone between the tendon and grout within the bond zone.

References

JGS (2012): *Design and Construction Standards for Ground Anchors, and Explanatory Notes*, The Japanese Geotechnical Society (JGS), 211P.

7. 4 Examples of Items Requiring Attention When Developing Technical Standards

7. 4. 1 Calculation of Required Anchor Force

The required anchor force for slope failure countermeasures shall be determined by considering both the shear-resistance increasing effect and the driving force reduction effect. On the other hand, for countermeasures against reactivated slow-moving landslides, the required anchor force is typically determined by accounting only for the restraining effect.

【Explanation】

The effects of ground anchors can be divided into the shear-resistance increasing effect, which increases shear strength by increasing the normal force on the slip surface, and the driving force reduction effect, which directly inhibits sliding through the tangential force on the slip surface. The calculation methods for the required anchor force P in anchor design are presented below, categorized into circular and non-circular slip surfaces. While these formulas reflect equations that consider both the shear-resistance increasing effect and driving force reduction effect, if only one of these effects is to be considered, they can be easily utilized by deleting the portion of the formula related to the other effect.

1) In the case of the simplified Bishop method (circular slide)

$$F_P = \frac{\sum \dfrac{c'b + (W - ub)\tan\phi'}{m_\alpha} + \dfrac{P\sin\theta\tan\phi'}{m_\alpha}}{\sum W\sin\alpha - P\cos(\theta + \alpha)}$$

$$P = \frac{F_P\sum T - \sum S}{F_P\cos(\theta + \alpha) + \sin\theta\tan\phi'/m_\alpha}$$

$$m_\alpha = \cos\alpha + \sin\alpha\tan\phi'/F_P$$

2) For the simplified Janbu method (non-circular slide)

$$F_P = f_0\frac{\sum\left[\dfrac{c'b + (W - ub)\tan\phi'}{n_\alpha}\right] + \dfrac{P\sin\theta\tan\phi'}{n_\alpha}}{\sum W\tan\alpha + Q - P\cos(\theta + \alpha)/\cos\alpha}$$

$$P = \frac{F_P\sum T - f_0\sum S}{F_P\cos(\theta + \alpha)/\cos\alpha + f_0\sin\theta\tan\phi'/n_\alpha}$$

$$n_\alpha = \cos^2\alpha\left(1 + \tan\alpha\tan\phi'/F_P\right)$$

Here, F_P: Design safety factor, c': Cohesion (kN/m²), b: Fragment width (m), W: Fragment self-weight (kN/m),
u: Interstitial water pressure (kN/m²), φ': Internal friction angle (°), P: Anchor force per unit width (kN/m)
α: Slip surface inclination angle (°), θ: Anchor installation angle (°)(angle to horizontal), f_0: Modification factor, $\sum T$: Denominator term in each stability analysis formula when P=0 (kN/m),
Q: Horizontal external force (kN/m), $\sum S$: Numerator term in each stability analysis equation when P=0, with safety factor F replaced by F_P (kN/m)

If the layer thickness of the slow-moving landslide mass is substantial (generally exceeding approximately 15

m along the anchor axial direction), the anchor force may not be sufficiently transmitted to the slip surface due to the arching effect of the rock mass within the slide mass and three-dimensional load distribution; therefore, its shear-resistance increasing effect cannot be expected. On the other hand, the driving force reduction effect is intended to hold back the moving mass when it attempts to slide along the slip surface, utilizing the tangential component of the anchor force on the slip surface. These individual mechanisms are represented by the forces illustrated in Figure 7.4.1.

In typical reactivated slow-moving landslides, the large moving layer thickness hinders the transmission of effective stress to the slip surface, and the gentle slope gradient often minimizes the shear-resistance increasing effect, designs typically prioritize safety by accounting only for the driving force reduction effect.

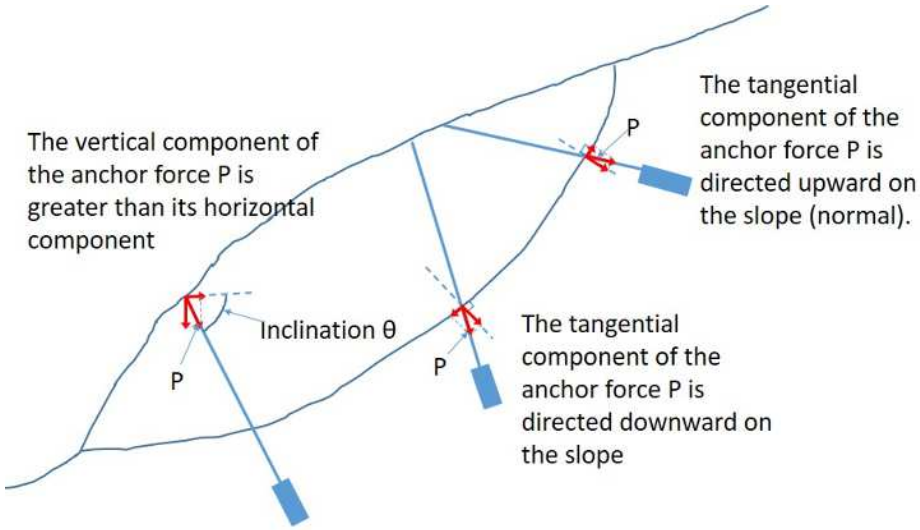


Figure 7.4.1 Mechanics of Ground Anchor Functions

Next, the design anchor force (T), which represents the anchor force borne per anchor, is calculated using the following formula.

$$T = \frac{m \cdot P}{n}$$

Hera, P: Required anchor force (kN/m)

m: Horizontal anchor spacing (m)

n: Number of anchor installation levels

T: Design anchor force (kN/anchor) (Anchor force borne per anchor)

7. 4. 2 Calculation of Fixed Anchor Length

In principle, the fixed anchor length shall be set between 3 m and 10 m, and it is determined by considering both the pull-out resistance between the ground and grout and the bond (restraining) force between the grout and tendon.

【Explanation】

In Japan, ground anchors are classified into two types based on their service life and the type of target structure, as shown in Table 7.4.1. The term "special conditions" referred to here denotes situations where cyclic loads constantly act, such as on the foundations of cable cranes, or environments under highly corrosive conditions, such as in hot spring areas or along coastlines.

Table 7.4.1 Classification of Ground Anchors Based on Service Life and Target Structural Conditions (Japanese Classification) (JGS, 2012)

Service Period / Structure Type	Less than 2 years	2 years or more
General Structures	Rank B	Rank A
Structures under special conditions	Rank A	

For bonded pre-stressed ground anchors, the required bond length between the soil and grout, and the required bond length between the tendon and grout, shall be compared. The longer of these two shall be adopted as the embedment length of the anchor body.

(1) Required bond length (L_a) between the ground and the grout

$$L_a = \frac{f_s \cdot T}{\pi \cdot d_A \cdot \tau}$$

Here, L_a: Required bond length between ground and grout (m)

f_s: Safety factor (Japanese standard values listed in Table 7.4.2)

T: Design anchor force (kN/unit)

d_A: Anchor body diameter (m)

τ: Per unit area circumferential friction resistance (kN/m²)

Table 7.4.2 Factors of Safety f_s for Ultimate Pull-out Capacity Adopted in Japan (JGS, 2012)

Anchor Classification		Safety Factor f_s
Rank B (Temporary Use)		1.5
Rank A	Non-seismic conditions	2.5
	Seismic conditions	1.5～2.0

In principle, the ultimate bond strength value shall be determined based on field tests (anchor investigation tests); however, if it is necessary to temporarily determine the fixed anchor length prior to these field tests, it is decided by referencing Japan's standard baseline values (Table 7.4.3). If there are existing standard baseline values for ultimate bond strength that correspond to the rock types of the target country, those local values should be prioritized and utilized. Table 7.4.3 Ultimate bond strength of anchors (Japan's standard baseline values, extracted for rock masses only)

Table 7.4.3: Ultimate Bond Strength of Anchors (Japanese Standard Values, Extracted for Rock Only) (JGS, 2000)

Type of Ground		Bond Strength (MN/m²)
Rock	Hard Rock*	1.5 – 2.5
	Soft Rock*	1.0 – 1.5
	Weathered Rock	0.6 – 1.0
	very stiff clay	0.6 – 1.2

A classification unique to Japan based on earthwork workability

In Japan, the distinction between hard rock and soft rock is determined as shown in the table below. Although the method for distinguishing between soft rock and weathered rock is ambiguous, it is practical to consider "Soft Rock I" as weathered rock in this context.

Table 7.4.4 Rock Classification in Earthwork (MLIT, Common Specifications for Civil Engineering Works, March 2011)

Name				Description	Summary
A	B	C			
Rock	Rock mass, Boulders			・Rock mass containing boulders, making excavation difficult and prone to clogging buckets, etc. ・Rock mass and boulders shall have a particle size of 7.5 cm or larger, with rounded ones classified as boulders.	Soil mixed with boulders, rock mass, crushed rock, and boulder-strewn riverbed
	Soft Rock	Soft Rock	I	・Tertiary rocks with weak consolidation. ・Extremely weathered and highly friable. ・Materials that can be separated by hand, with cracks spaced approximately 1–5 cm apart, and Tertiary rocks with good consolidation. ・ Significantly weathered with some discoloration, easily fractured by light impact, prone to separation, with crack spacing approximately 5–10 cm.	Elastic wave velocity 700 – 2800m/s
			II	・Tuffaceous and firmly consolidated. Significantly weathered along the grain. ・ Fracture spacing of approximately 10–30 cm, allowing separation with light impact; heterogeneous, hard interbedded layers that can be easily separated along bedding planes.	
	Hard Rock	Medium-hard rock		Limestone, porous andesite, etc., possessing considerable hardness even if not particularly dense. Not significantly weathered. Hard rock with cracks spaced approximately 30–50 cm apart.	Elastic wave velocity 2000 – 4000m/s
		Hard Rock	I	Granite, gneiss, or similar rocks showing no alteration whatsoever. Fractures spaced approximately 1m apart with tight bonding. Suitable for yielding hard, high-quality stone.	Elastic wave velocity 3000m/s or higher
			II	The hardest rock types rich in carbonate minerals, such as limestone or breccia. Fresh, unweathered condition. Few fractures with tight bonding.	

Table 7.4.5 Guidelines for Ground Classification Using Heavy Equipment Workability and In-Situ Elastic Wave Velocity (Compiled from JRA (1986) and JRA (2009))

Ground Classification	Criteria based on construction feasibility with heavy machinery, JRA (1986)	Ground elastic wave velocity, JRA (2009a)
Soil	Excavation possible using a 21-ton class bulldozer's dozer blade	Approximately 700 m/s
Soft Rock	Excavation using a 21-ton class bulldozer's dozer blade is difficult Ripper work possible using a 32-ton class bulldozer	Approximately 2.0 km/s or less
Hard Rock	Ripper work using a 32-ton class bulldozer is difficult	Approximately 2.0 km/s or more

Figure 7.4.2 summarizes the ultimate bond strength obtained from anchor investigation tests in Japan, categorized by geological age (JAA, 2013). Notably, some cases exhibit values significantly lower than the standard baseline values listed in Table 7.4.3. Based on these test results, the standard baseline values for the target country are established, or existing local standards are reviewed and revised.

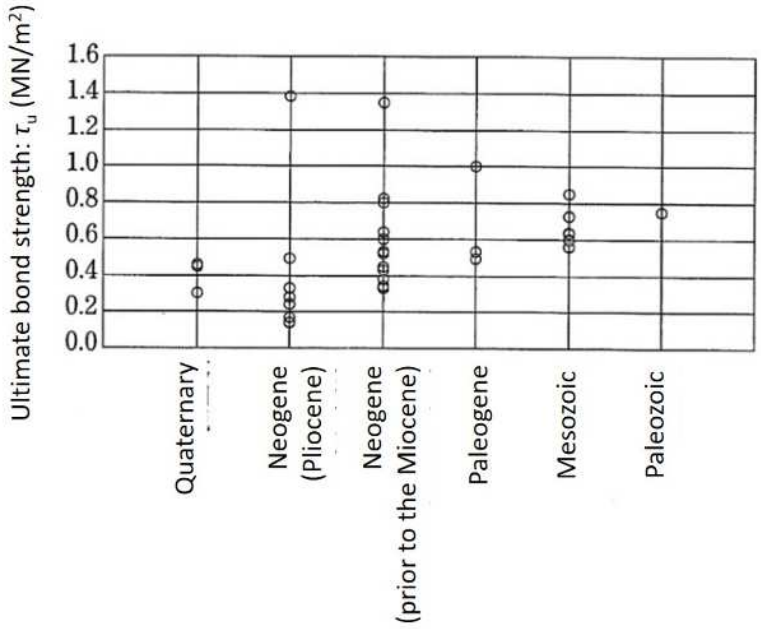


Figure 7.4.2 Relationship between Geological Age and Ultimate Bond Strength in Japan (JAA, 2013)

For reference, examples of ultimate bond strength τ and factor of safety f_s adopted in various countries are shown in Tables 7.2.5 to 7.2.7. The standard baseline values for the target country shall be established by referencing these international practices.

(Note: This table is to be inserted here when actually drafting the standard manual for the target country.)

(2) Required bond length (L_{sa}) between tendon and grout

$$L_{sa} = \frac{T}{\pi \cdot d_s \cdot \tau_b}$$

Here, L_{sa}: Required bond length between tendon and grout (m)

T: Design anchor force (kN/unit)

d_s: Tendon diameter (m)

τ_b: Allowable bond stress between tendon and grout (kN/m²)

The Japanese standard values for the allowable bond stress between tendon and grout are shown in Table 7.4.6. (This is quoted from JGS (2012), but the original source is the Standard Specifications for Concrete Structures published by the Japan Society of Civil Engineers (JSCE, 1976). If similar standards exist in the relevant country, those should be adopted.)

Table 7.4.6: Allowable Bond Stress Between Grout and Tendons (MN/m²) (JGS, 2012)

Classification	Design Strength of Grout / Type of Tensile Material	18	24	30	40 or more
Grade B	PC Steel Wire PC Steel Bar PC Steel Strand Multiple PC Steel Strand	1.0	1.2	1.35	1.5
	Deformed PC Steel Bar	1.4	1.6	1.8	2.0
Grade A	PC Steel Wire PC Steel Bar PC Steel Strand Multiple PC Steel Strand	—	0.8	0.9	1.0
	Deformed PC Steel Bar		1.6	1.8	2.0

(3) Determining Fixed Anchor Length

The required bond length between the ground and grout L_a is compared with the required bond length between the tendon and grout L_{sa}, and the longer of the two shall be adopted as the fixed anchor length. In principle, the fixed anchor length shall be set between 3 m and 10 m. If the calculated fixed anchor length exceeds the upper limit of 10 m, engineering modifications such as reducing the design anchor force must be considered. Furthermore, if the calculation result for the required bond length is shorter than 3 m, the fixed anchor length shall be set to 3 m.

References

JGS (2012): *Design and Construction Standards for Ground Anchors, and Explanatory Notes*, The Japanese Geotechnical Society (JGS), 211P.

JSCE (1976): *Standard Specifications for Concrete Structures -Design Edition*, Japan Society of Civil Engineers (JSCE).

JAA (2013): *Ground anchor design/ construction instruction manual*, Japan Anchor Association (JAA), 263P.

JRA (1986): *Guidelines for Highway Earthwork: Soil Investigation*, Japan Road Association (JRA)

7. 4. 3 Design of Reaction Structures

Because the ground surface where ground anchors are installed is generally a soil layer, the anchor head at the surface is prone to settlement due to the tensioning of the tendon by the landslide sliding force. To prevent this settlement and ensure that the effects of the ground anchors are fully exerted, a reaction structure shall be installed at the anchor head.

【Explanation】

1. Overview

Reaction structures include plates and crib works, which shall be selected by thoroughly considering the slope conditions, anchor parameters, constructability, economic efficiency, maintenance, and landscape. Furthermore, the stability conditions shall be investigated as follows according to the specific type of reaction structure and slope conditions:

(1) Stability Against the Bearing Capacity of the Ground

Although the bearing capacity of the ground supporting the reaction structures should ideally be determined through field load tests, when calculating it, Terzaghi's bearing capacity formula is widely used in Japan.

(2) Stability Against Bending Failure and Punching Shear Failure

The structural stability against bending failure of reaction structures is commonly evaluated by modeling them as beams supported by the anchor fixation points. When fixing multiple anchors to a single reaction structure, the cross-sectional thickness and the required steel area are determined by calculating the sections between adjacent anchors as a continuous beam and the end portions as a cantilever beam. In this structural analysis, the load is treated as a uniformly distributed load.

Furthermore, when concrete grillage beams are utilized as reaction structures, the internal cross-sectional forces shall be evaluated by treating the subgrade reaction as a uniformly distributed load and assuming that the anchor force acts in both the vertical and horizontal beam directions.

2. Design of Member Cross-sections

When the concrete crib works are utilized as reaction structures, the *Design Manual for Concrete Crib Works* (JSPA, 2013) shall be followed. The following section describes the design of member cross-sections when independent reaction structures are employed as reaction structures.

(1) Analysis of Bending Moment and Shear Force

Bending moments and shear forces shall be examined assuming that a uniformly distributed subgrade reaction equal to the design anchor force is acting. The analytical model for verification is fundamentally evaluated as a cantilever beam. The methodology for calculating the internal cross-sectional forces for the beam model shown in Figure 7.4.3 is presented below:

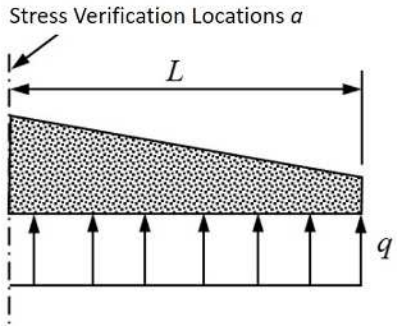


Figure 7.4.3 Load Diagram

1) Ground Reaction Force

$$Q = \frac{T_d}{A}$$

Here, Q: Ground Reaction Force (N/mm²). T_d: Design Anchor Force (N), A: Area of the Pressure Plate (mm²)

2) Uniformly Distributed Load

$$q = Q \times b$$

Here, q: Uniformly Distributed Load (N/mm), b: Design Beam Width (mm)

3) Shear force at the base (line a)

$$S = q \times L$$

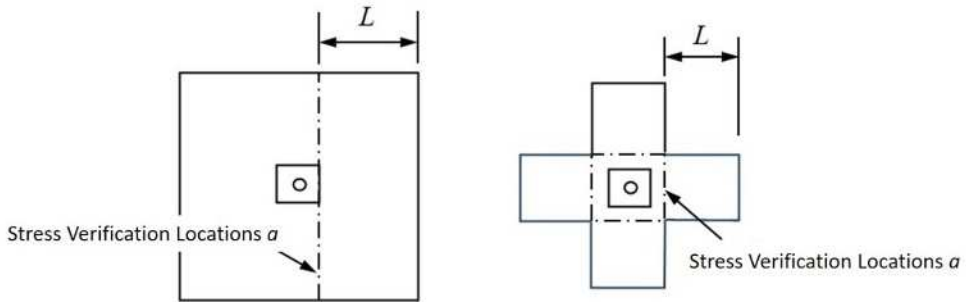
Here, S: Shear force at the base (line a) (N), L: Span (mm)

4) Bending moment at the base (line a)

$M = 0.5 \times q \times L^2$

Here, M: Bending moment at the base (line a) (N·mm)

The span is defined as the length from the base to the tip, and the verification position for the cross-section is set at the base. Examples of the span and the cross-section verification position are illustrated in Figure 7.4.4.



(a) Rectangular Reaction Structure **(b) Cross-shaped Reaction Structure**

Figure 7.4.4 Example of Span and Cross-section Verification Position

(2) Analysis of Bearing Pressure and Push-Through Shear Force

Bearing force and punching shear force shall be examined assuming that they act directly from the anchor. An example of calculating the bearing stress is presented below (see Figure 7.4.5).

$$\sigma_c' = \frac{T_d}{A_a}$$

Here, σ'_c: Bearing pressure stress (N/mm²), A_a: Effective bearing area of the bearing plate = $b_b^2 - \pi D^2/4$ (mm²),

T_d: Design anchor force (N), b_b: Width of the bearing plate (mm), D: Diameter of the anchor hole (mm)

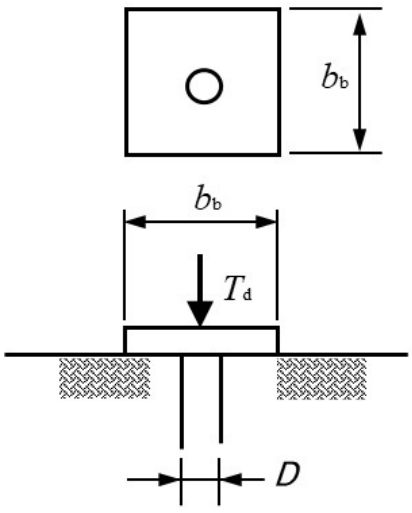


Figure 7.4.5 Parameters for Calculating Bearing Stress

The calculation formula for punching shear stress is shown below (see Figure 7.4.6).

$$\tau_p = \frac{T_d}{U_p \cdot d}$$

Here, τ_p: Punching shear stress (N/mm²), U_p: Design cross-section perimeter = $\pi \cdot d + 4b_b$ (mm)

T_d: Design anchor force (N), d: Effective height (mm)

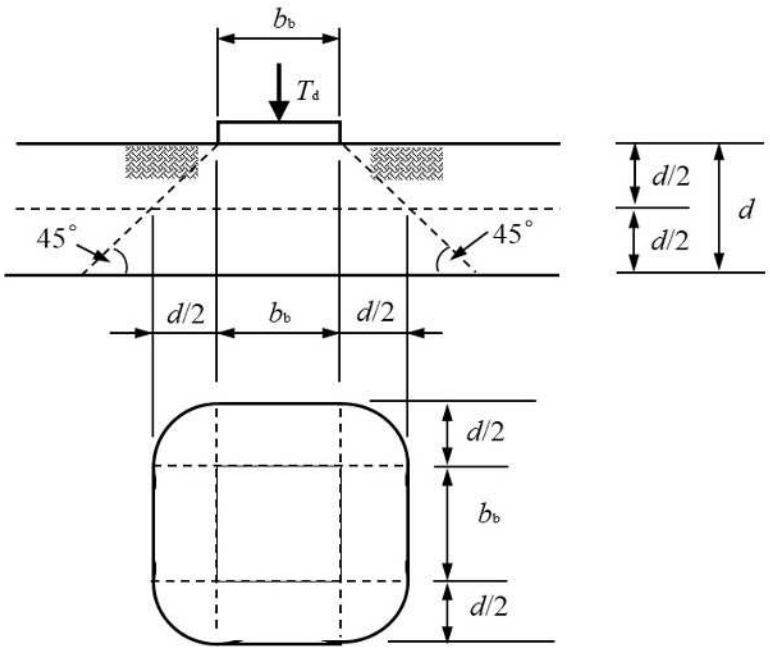


Figure 7.4.6 Parameters for Calculating Punching Shear Stress

References

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JFCA (1987): *Technical Standards and Commentary for Forest Conservation: Slow-moving Landslide Prevention Edition*, Japan Forest Conservation Association (JFCA).

JFCA (2013): *Technical Standards and Commentary for Forest Conservation: Slow-moving Landslide Prevention Edition*, Japan Forest Conservation Association (JFCA), 204p.

JSPA (2013): *Design and Construction Guidelines for Concrete Crib Works for slope*, Japan Slope Protection Association (JSPA), 99p.

8. Design of Slope Reinforcement Works

8. 1 Overview of Slope Reinforcement Works

8. 1. 1 Characteristics of Slope Reinforcement Works

Slope reinforcement works involve constructing grouted nails by inserting steel nails such as steel bars into cut slopes, natural slopes, or existing embankment slopes, then injecting cement slurry or similar grout around their periphery. This integrates the grouted nail with the natural slope to stabilize it. This method constrains ground deformation by utilizing the passive resistance force generated in the grouted nails as the ground deforms, thereby stabilizing the slope. Its key features are a simple structure and the ability to be constructed in a short period.

The NATM method as tunnel support, vertical grout-grout bolt reinforcement for tunnel portals, as well as methods employed for underpinning and excavation support, all share a similar structural principle.

8. 1. 2 Structure and Types of Slope Reinforcement Works

Figure 8.1.1 shows the basic structure of slope reinforcement works. Composed of grouted nails and facing materials (including nail head assembly), it features a simpler structure compared to ground anchoring techniques and is economical, leading to its widespread adoption as a slope stabilization measure.

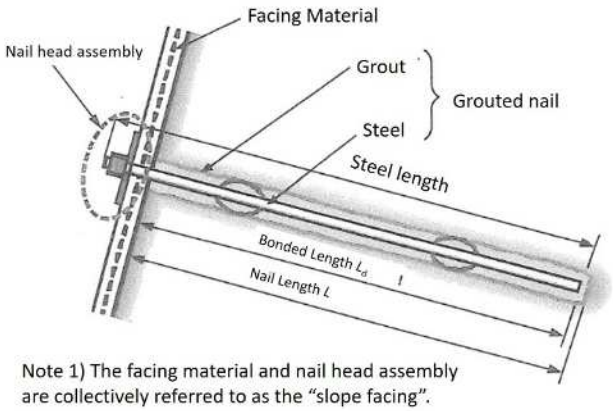
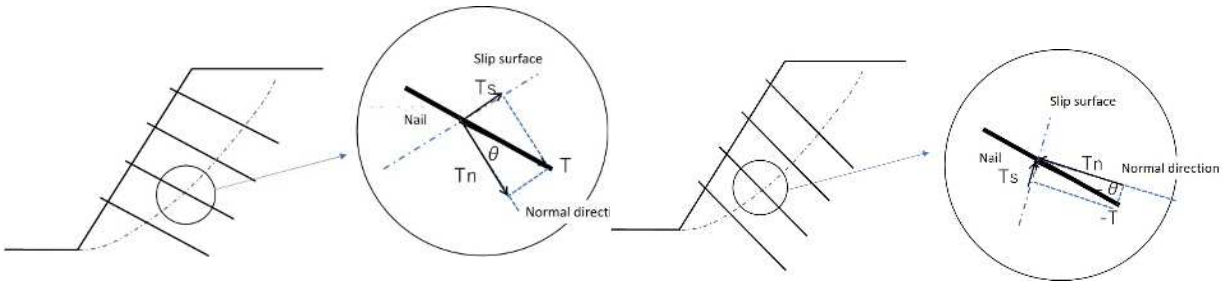


Figure 8.1.1 Basic Structure of Slope Reinforcement Works (JGS, 2011)

As shown in Figure 8.1.2, slope reinforcement works can be classified into "tension reinforcement," "shear/bending reinforcement," and "compression reinforcement" based on the angle formed between the grouted nails and the slip plane. Until the early 1990s, cut-and-fill reinforced earth using structural steel sections as reinforcement was designed as "shear/bending reinforcement."

However, due to limited research results and the tendency for large deformations, designs now frequently employ "tension reinforcement." Consequently, the concept of tension reinforcement has become established in the design of slope reinforcement works.



(a) Tensile reinforcement **(b) Compressive reinforcement**

Figure 8.1.2 Differences in reinforcement effect based on installation angle

"Tension reinforcement" applies when the grouted nail axis is above the normal to the slip surface within the

stable layer, while “compression reinforcement” applies when the grouted nail axis is below the normal to the slip surface within the stable layer, resulting in a state where the compressive axial force in the grouted nail dominates. A mechanism similar to compression reinforcement is the support mechanism of structures using foundation piles. Compression reinforcement has not gained widespread adoption domestically due to limited research outcomes and complex construction management.

On the other hand, when the shear plane and the grouted nail are nearly perpendicular, both shear reinforcement and bending reinforcement effects are exerted, as shown in Figure 8.1.3.

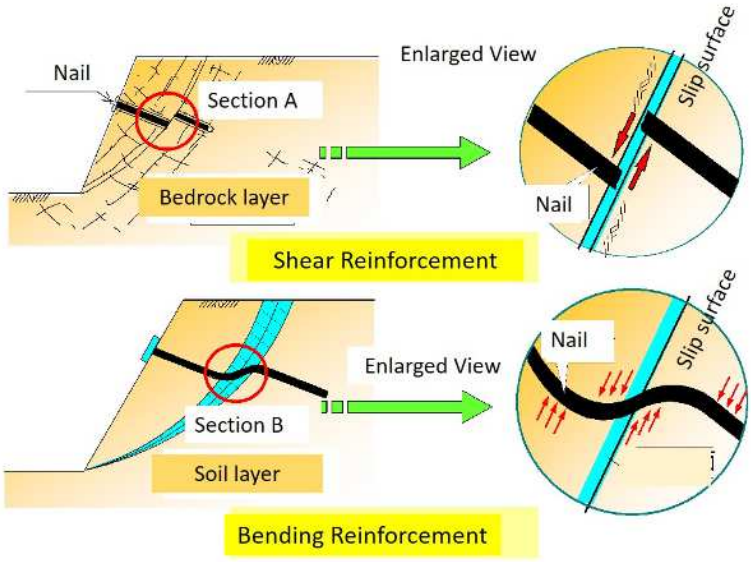
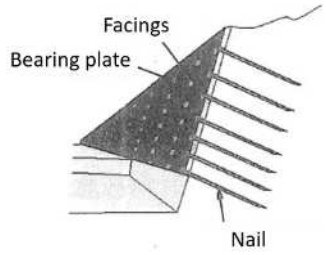
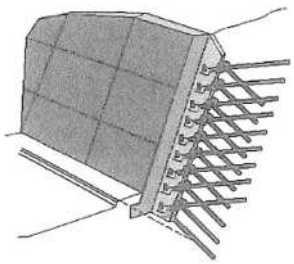
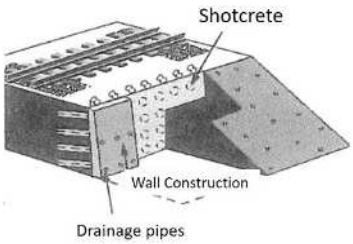


Figure 8.1.3 Difference between Shear Reinforcement Effect and Bending Reinforcement Effect (JGS, 2011)

Table 8.1.1 shows examples classified by the slenderness ratio of the grouted nail. Generally, construction methods using steel bars as reinforcing elements belong to the “nailing method.” On the other hand, methods using grouted nails with a small slenderness ratio and high rigidity are called “dawling methods.” Methods using grouted nails with a slenderness ratio intermediate between these “nailing methods” and “dawling methods” belong to the “micropiling method,” which is mainly adopted in Japan for reinforcing existing retaining walls and bridge piers. In Japan, the term “rebar insertion works” is commonly used for reinforcement works. However, overseas, the term “Soil Nailing” is often used to describe this method, including the Japanese rebar insertion works. This chapter focuses on “soil nailing,” a specialized slope stabilization technique.

Table 8.1.1 Types of Slope Reinforcement Works Based on Slenderness Ratio of Grouted Reinforcements (JGS, 2011)

Method	Nailing	Micro Piling	Doweling
Schematic Diagram			
Overview	This method involves placing grouted nails with a high aspect ratio and low bending stiffness into the ground to enhance ground stability primarily through the tensile resistance of the grouted nail. Representative methods include soil nailing and earth nailing.	A method that reinforces the ground with intermediate bending stiffness and cross-sectional area compared to nailing and doweling, utilizing the tensile resistance, bending resistance, and compressive resistance of the micropiles. Examples include the root pile method.	This method involves placing grouted dowls with a low slenderness ratio and high rigidity into the ground. It enhances slope stability not only through the tensile resistance of the grouted dowls but also through its bending and compressive resistance. Examples include the Radish Anchor method.

Figures 8.1.4 to 8.1.5 show examples of the primary slope reinforcement works (soil nailing) used in Japan.



Figure 8.1.4 Shotcrete crib works Construction Using Soil nailing



Figure 8.1.5 Head-Linked Soil nailing

References

JGS (2011): *Manual for Design and Construction of Slope Reinforcement Method*, The Japanese Geotechnical Society (JGS), PP.5-17.

8. 2 Key Issues and Actions to Take in Internationalization

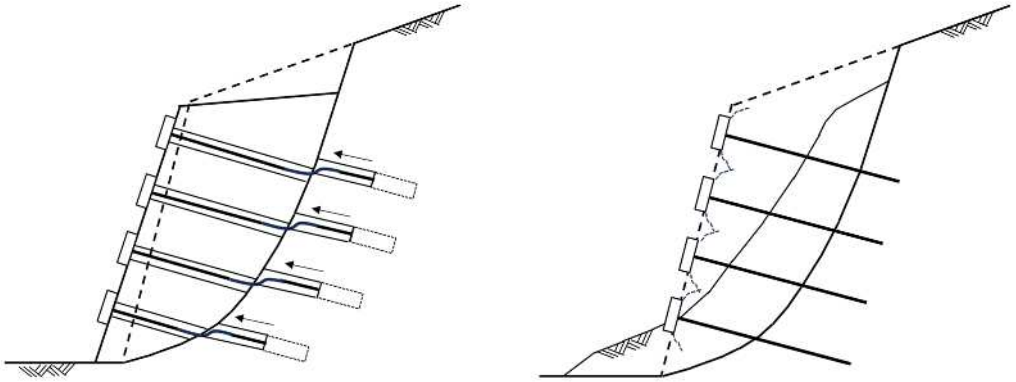
8. 2. 1 Design Methods Incorporating Japan's Unique Slope Facing Effects

(1) Issues

In Japan, designs for soil nailing incorporate the effects of slope facing. However, overseas, design methods incorporating slope facing effects are rare. Coefficients related to slope facing effects also employ calculation methods unique to Japan.

(2) Explanation

Figure 8.2.1 shows failure patterns of soil nailing in the road sector. Figure 8.2.1 (a) depicts a pattern where the grouted nails is pulled out. Figure 8.2.1 (b) shows a pattern where the grouted nails remains in the stable layer while the displaced unstable layer is pulled out. Therefore, in soil nailing, it is necessary to prevent the occurrences of these two patterns.



(a) Failure due to grouted nails pullout

(b) Failure due to unstable layer pullout (failure of soil surrounding grouted nails)

Figure 8.2.1 Failure patterns of soil nailing

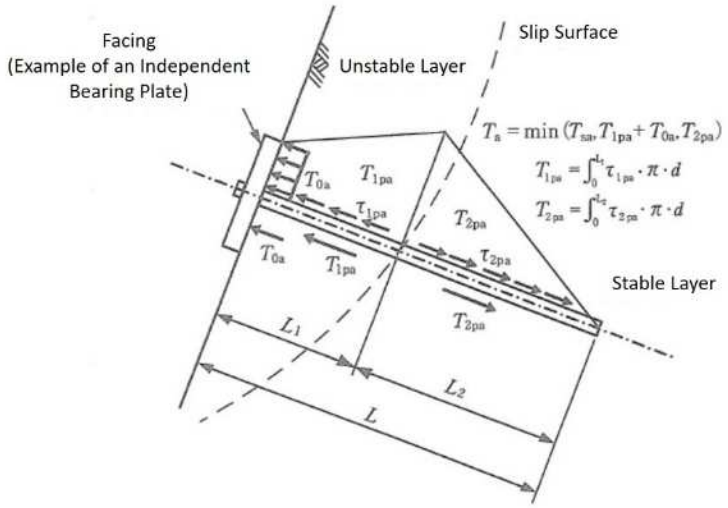


Figure 8.2.2 Effect of Slope Facing (Conceptual Diagram of Grouted Nail Resistance) (JGS, 2011)

Figure 8.2.2 illustrates the concept of slope facing effectiveness (grouted nail resistance concept diagram). When the allowable pull-out resistance around the grouted nail on the unstable layer side is T_{1pa} and the allowable pull-out resistance around the grouted nail on the stable layer side is T_{2pa}, the total allowable pull-out resistance

on the unstable layer side is the sum of the allowable pull-out resistance T_{1pa} around the grouted nail on the unstable layer side and the allowable bearing resistance T_{0a} provided by the slope facing. Furthermore, the total allowable pull-out resistance on the stable layer side is equal to the allowable pull-out resistance T_{2pa} around the grouted nail.

Therefore, for the failure pattern shown in Figure 8.2.1(a), T_{2pa} must be sufficiently large relative to the slip force. For the failure pattern shown in Figure 8.2.1(b), $T_{1pa} + T_{0a}$ must be sufficiently large relative to the slip force.

Therefore, the allowable tensile resistance T_a of the grouted nail to be considered in the design is the minimum value among the allowable tensile strength T_{sa} of the steel nail, the total allowable pull-out resistance T_{2pa} on the stable layer side, and the total allowable pull-out resistance $T_{1pa} + T_{0a}$ on the unstable layer side.

$$T_a = \min(T_{sa},\ T_{1pa}+T_{0a},\ T_{2pa}) \quad \cdots(1)$$

Source: *Manual for Design and Construction of Slope Reinforcement Method* (JGS, 2011)

Here, the coefficient "facing effect factor: μ" corresponding to the type of facing material is used to evaluate $T_{1pa} + T_{0a}$ and calculate the allowable tensile force T_a of the grouted nail.

$$T_{1pa}+T_{0a} = 1/(1-\mu)\cdot T_{1pa} \quad \cdots(2)$$

$$T_{1pa}=L_1\cdot t_a \quad \cdots(3)$$

μ: Facing effect factor ($=T_{0a}/(T_{1pa}+T_{0a})$)

However, when μ = 1, verification using Equation (2) is not required.

L_1 : Effective bonded length of displaced unstable layer

t_a : $\min(t_{pa},\ t_{ca})$

t_{pa}: $\tau_{pa}\cdot\pi\cdot D/F_s$

(τ_{pa}: Bond strength between ground and grout, D: Borehole diameter, F_s: Boned strength safety factor)

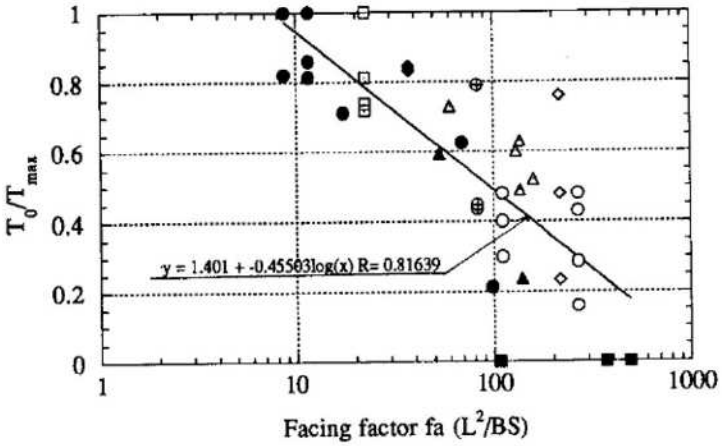
t_{ca}: $\tau_0\cdot\pi\cdot d$

(τ_0: Allowable bond strength between steel nail and grout, d: Steel nail diameter)

Table 8.2.1 shows the representative "facing effect factor μ" for slope facings. Figure 8.2.3 summarizes the experimental results and field measurement results that formed the basis for Table 8.2.1. The horizontal axis shows the "facing effect factor," which is the square of the grouted nail length divided by the slope facing pressure-bearing width per grouted nail member and the grouted nail spacing. The vertical axis, "facing effect factor μ," shows that it depends on the length of the grouted nail, the pressure-bearing area (width) of the slope facing, and the spacing between grouted nail.

Table 8.2.1 Facing Effect Factor μ for Major Slope Facing (NEXCO, 2011)

Slope Facing Type	μ	Remarks
Vegetation Slope	0	
Shotcrete Works	0.2～0.6	
Concrete Crib Works	0.7～1.0	
Retaining Walls	1.0	Continuous Slab Type Slope Facings



L: Length of grouted nail, S: Spacing between grouted nails, B: Width of slope facing $(=A^{1/2})$, A: Pressure-bearing area, T_0: Tensile force acting on the slope facing, T_{max}: Maximum tensile force

Figure 8.2.3 Relationship between facing factor fa and facing effect factor μ (= T_0/T_{max}) (Muramatsu et al., 1994)

In overseas nailing methods, reaction structures may not be used, and grouted nail lengths often exceed 10m. Consequently, overseas design software frequently does not account for the effectiveness of slope facings.

Figure 8.2.4 explains the internal forces generated in grouted nail connected to rigid wall structures like retaining walls, as described in the Federal Highway Administration (FHWA, 2015) manual. It states that slope facings impose tensile force on the grouted nail, considering the effect of slope facings on reinforcement force generation similarly to Japan—as a tensile force increase effect.

Figure 8.2.5 shows the tensile force distribution when grouted nail is placed behind a rigid wall surface, such as in retaining structures. The maximum tensile force acting on the grouted nail occurs at a depth of 0.3 to 0.4 times the wall height, indicating that the tensile force at the slip surface location is not the maximum tensile force (FHWA, 2015).

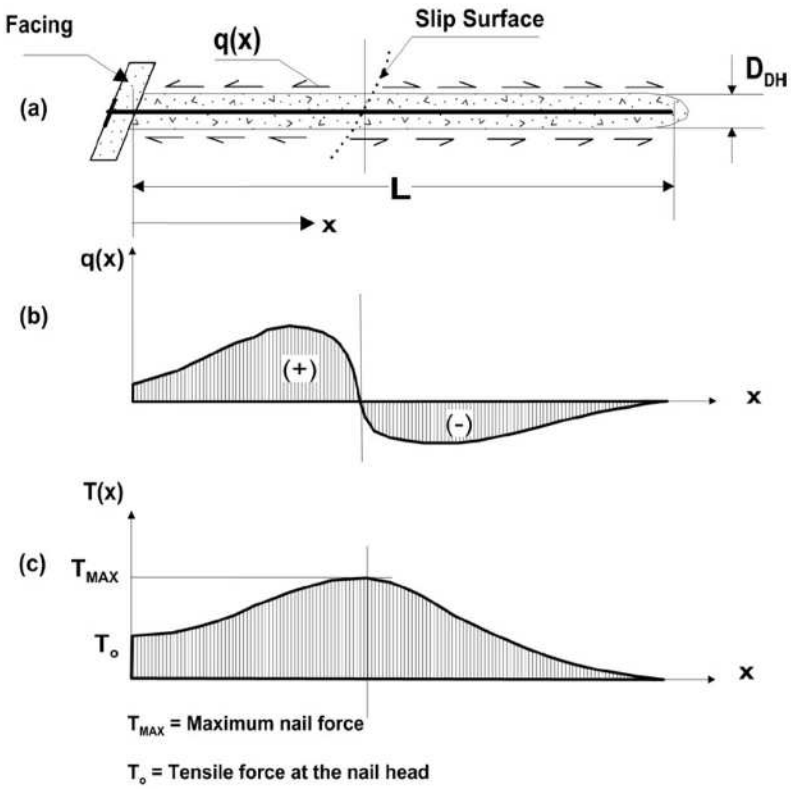


Figure 8.2.4 Distribution of Internal Forces in Soil Nails (FHWA, 2015)

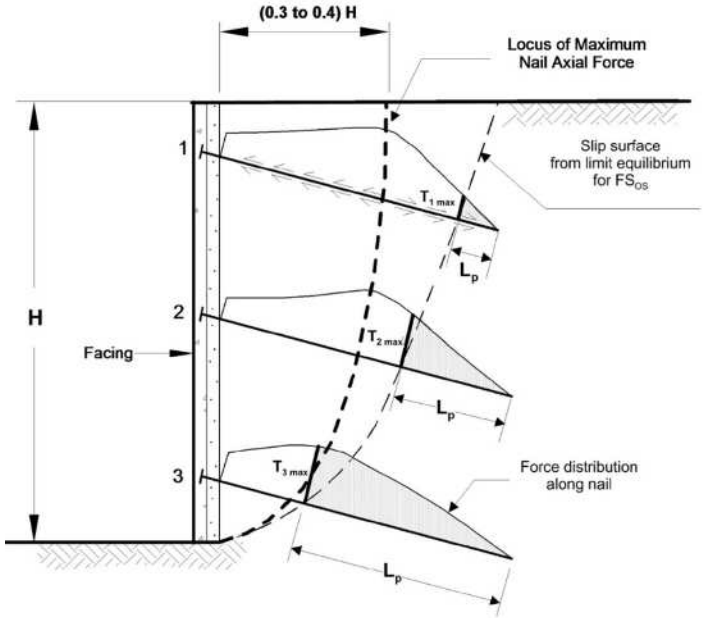


Figure 8.2.5 Distribution of Maximum Tensile Force in Soil Nails Combined with Rigid Wall Panels (FHWA, 2015)

Figure 8.2.6 shows the distribution of maximum nail tensile force T_{max} compiled from field measurement results by Bryen et al. (1998) (FHWA, 2015). Figure 8.2.7 shows the distribution of head tensile force T_0 compiled from field measurement results by Bryen et al. (1998) (FHWA, 2015).

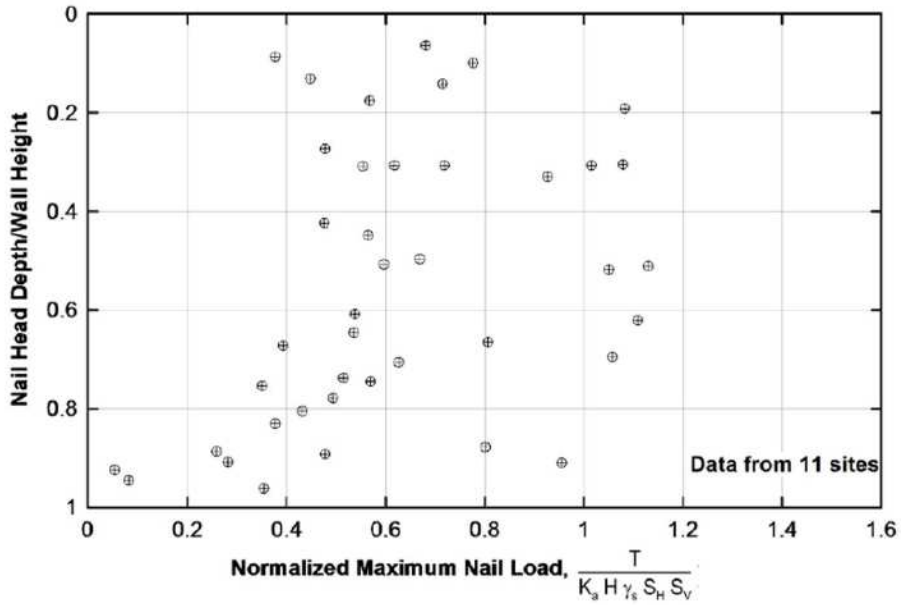


Figure 8.2.6 Field measurement results for maximum nail tensile force T_{max} (FWHA, 2015; original source: Bryen et al., 1998)

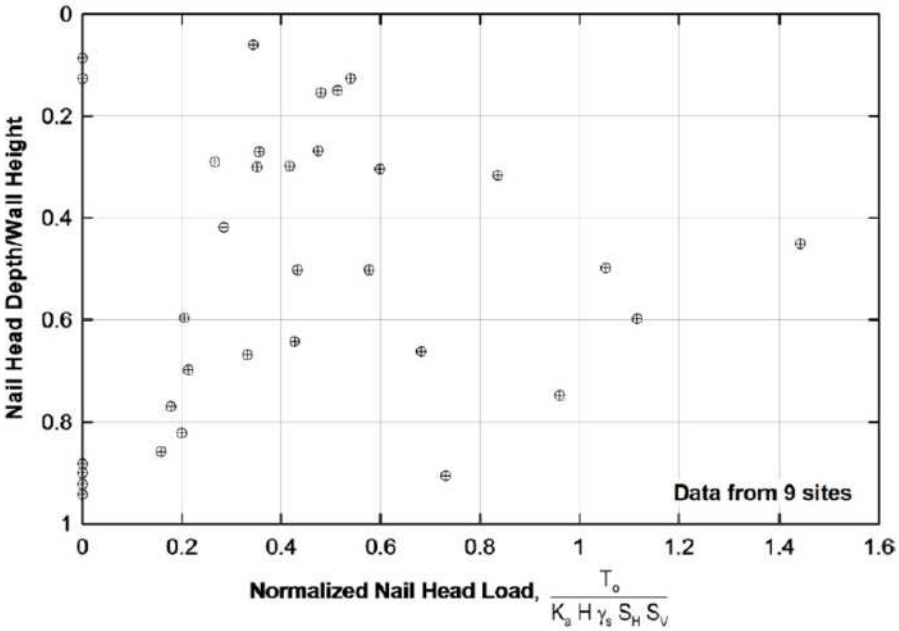


Figure 8.2.7 Field measurement results of the head tensile force T_0 of reinforcing bars (FWHA, 2015; original source: Bryen et al., 1998)

Cloterre (2002) proposed determining the tensile force T_0 acting on the head of the reinforcing element affected by the wall surface using Equation (1), based on the field survey results of Bryen et al. (1998) (FHWA, 2015).

$$T_0 = T_{max}(0.6 + 0.057(S_{max} - 3)) \quad \cdots (1)$$

Here, S_{max}: Maximum value of vertical and horizontal spacing of soil nail (F_t)

For soil nail spacing of 1.5 m (5 ft), $T_0 = 0.7T_{max}$.

Sile (2014) states that the design software SNAP-2 calculates the value derived from Rankine earth pressure as

the force acting on the slope facing and verifies whether this calculated value (load) exceeds the calculated resistance value (FHWA, 2015). However, Chapter 6 “Design of Soil Nail Walls” in FHWA (2015) states that the above equation (1) takes precedence in the design procedure (FHWA, 2015).

(3) Actions to Take

When using relatively short reinforcing elements (maximum 5 to 6 m) for soil nailing such as in Japan, use Japanese design software after understanding the logical basis for the facing effect factor. When designing long-span soil nailing exceeding 10 m overseas, it is necessary to use corresponding overseas design software.

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Muramatsu, M., Naruse, T., Mori, S. (1994): Design Methods for Slope Protection Structures in the Soil Nailing (Part 2), The 49th Annual Meeting of the Japan Society of Civil Engineers, pp.1622-1623.

NEXCO (2007): *Design and Construction Guidelines for Cut Slope Reinforcement Method*, Nippon Expressway Company (NEXCO), PP.50-51.

FHWA (2015): *Soil nail walls reference manual*, FHWA-NHI-14-007, 385p.

Byrne, R.J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G. (1998): *Manual for Design and Construction Monitoring of Soil Nail Walls*, Report No. FHWA-SA-96-69R, Federal Highway Administration, Washington, DC.

Cloterre (2002): Additif 2002 aux recommandations Clouterre 1991, (Trans: Addenda to Recommendations Clouterre 1991), in French, Presses de l’Ecole Nationale des Ponts et Chaussées, Paris, France.

Siel, B.D. (2014): SNAP-2 (Soil Nail Analysis Program) User’s Manual, Report No. FHWA-OTS-XXX, FHWA Resource Center, Federal Highway Administration, Lakewood, CO.

8. 2. 2 Mechanical Interpretation of Driving Force Reduction Effect in Earth Reinforcement Performance

(1) Issues

In Japanese design practices for tensile soil nailing, both the shear resistance increasing effect (ΔR_c) and driving force reduction effect (ΔR_w) due to tensile force are incorporated into the numerator of the stability analysis equation. Conversely, overseas approaches sometimes reduce the driving force reduction effect in the denominator of the stability analysis equation. Furthermore, while Japan interprets the driving force reduction effect of reinforced soil as a force, Western countries often consider it as a strength. The difference between Japanese and overseas design methods also presents an issue.

(2) Explanation

In general slope stability assessments, the ratio of shear strength to sliding force is used as the safety factor. There is also a concept that the shear resistance increasing effect enlarge shear strength by enhancing friction resistance, while the driving force reduction effect decreases sliding force (JGS, 2011).

Therefore, in this case, as shown in Equation (1), the shear-resistance increasing effect of the soil nailing is placed in the numerator term, while the restraint effect ΔR_w (= $T_r \cdot \cos\beta$) due to the tensile force T_r is placed in the denominator term as a factor that reduces the slip force.

$$F_s = \frac{S + \Delta R_c}{Q - \Delta R_w} \qquad \cdots(1)$$

Here: Q: Shear force of soil blocks along the slip surface (kN/m) when unreinforced, S: Shear strength of soil mass (kN/m) when unreinforced, β: Angle between the slip surface and the soil nail (°)

In Japan, when evaluating the reinforcement effect of grouted nails, researchers refer(e.g., Jewell(1980)) to findings indicating that the sum of the shear-resistance increasing effect ΔR_c $(=T_r \cdot \sin\beta \cdot \tan\varphi)$ and the driving force reduction effect ΔR_w $(=T_r \cdot \cos\beta)$ caused by tensile force T_r can be evaluated as the component of the increase in cohesion of the soil mass in which the grouted nail is placed. Consequently, the mainstream approach in Japan is to incorporate both the shear-resistance increasing effect (ΔR_c) and the driving force reduction effect (ΔR_w) caused by tensile force into the design (see Equation (2)).

Furthermore, when the driving force reduction effect $\Delta R_w > Q$, the solution becomes negative. Due to reasons such as high sensitivity to the safety factor when grouted nail are placed, in practice, it is common to evaluate using Equation (2), which generally provides a less sensitive and safer assessment (JGS, 2011, pp.98).

$$F_s = \frac{S + \Delta R_w + \Delta R_c}{Q} \qquad \cdots(2)$$

In papers concerning overseas nailing methods, some adopt Equation (1) while others adopt Equation (2). In cases adopting Equation (2), some place the driving force reduction effect in the molecular term, based on the concept that it is the strength of the grouted nail rather than forces such as tensile force.

Tables 8.2.2 to 8.2.4 show the approaches to evaluating the effectiveness of countermeasures in slope analysis software used in Europe and the United States. The globally widely used slope analysis software “SLOPE/W: Geo-Slope Ltd., Canada” does not consider the shear-resistance increasing effect. For the driving force reduction

effect, when adopting Equation (2), it assumes division by the safety factor. When calculating the slope safety factor, it allows selection between Equation (1) and Equation (2).

SLIDE: Canada Rocscience adopts Equation (2) for soil nailing (passive anchors), where the driving force reduction effect is considered as the strength of the grouted nail, as resistance is exerted in response to slope deformation. For ground anchors (active anchors) that apply prestressing force in advance or rock bolts capable of applying tension loads, Equation (1) is considered preferable. “SLOPE: Italy, Geostru” does not consider the shear-resistance increasing effect and instead employs a unique driving force reduction effect.

Table 8.2.2 Safety Factor Concepts in Slope Analysis Software Used in Western Countries (1)

Software Name	Slope/W	
Provider	Geo-SLOPE (transferred to Bentley in 2021)	
Location	Canada	
Concept of Safety Factor	Approach where reinforcement force reduces slip force (Most common)	$FS = \frac{\sum resisting\ forces}{\sum driving\ forces - \sum anchor\ forces}$
	Approach where reinforcement force increases resistance force	$FS = \frac{\sum resisting\ forces + \sum anchor\ forces}{\sum driving\ forces}$ In this case, the numerator value must be divided by the safety factor

Table 8.2.3 Safety Factor Concepts in Slope Analysis Software Used in Western Countries (2)

Software Name	SLIDE2	
Provider	Rocscience	
Location	Canada	
Concept of Safety Factor	Active Support Type -Grouted Tieback -Tensioned Cable -Rock bolt	$F = \frac{resisting\ force + T_N \tan\phi}{driving\ force - T_S}$ T_n: Normal component T_s: Shear component
	Passive Support Type -Soil nail -Geotextile	$F = \frac{resisting\ force + T_N \tan\phi + T_S}{driving\ force}$

Table 8.2.4 Safety Factor Concepts in Slope Analysis Software Used in Western Countries (3)

Software Name	Slope	
Provider	GeoStru	
Location	Italy	
Concept of Safety Factor	Active anchor Type	$FS=R_d/[E_d-\Sigma_{i,j}R_{i,j}\cdot(1/\cos\alpha_i)]$ R_d: Numerator term of stability analysis E_d: Denominator term of stability analysis R_{ij}: Horizontal anchor strength α_i: Angle between horizontal component of tensile force and slip plane normal
	Passive anchor and nails	$FS=[R_d+\Sigma_{i,j}R_{i,j}\cdot(1/\cos\alpha_i)]/E_d$

The U.S. Soil Nail Walls Reference Manual (FHWA, 2015) specifies the limit state design method as the design approach for soil nails. These three design software packages support the limit state design method compliant with

Eurocode 7.

(3) Actions to Take

Although the fundamental concept differs, the design method prevalent in Japan (Type 2) ultimately results in the same design approach as Rocscience's method. Adopting the design philosophy of utilizing the strength of grouted nails, as Rocscience does, reduces the mechanical issues in Japanese design methods. Therefore, using Japanese design software directly overseas poses no significant issues.

References

JGS (2011): *Manual for Design and Construction of Slope Reinforcement Method*, The Japanese Geotechnical Society (JGS), PP.89.

FHWA (2015): *Soil nail walls reference manual*, FHWA-NHI-14-007, 385p.

GEO-SLOPE (2015): Stability modeling with SLOPE/W.

GeoStru (2021): Slope stability analysis, https://www.geostru.eu/blog/2016/06/13/slope-stability-analysis/, Accessed 18 Jun 2021.

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Jewell, R. A. (1980): Some effects of reinforcement on the mechanical behaviour of soils, P h. D. Thesis, C ambridge Univ.

8. 2. 3 Reinforcement Effect of Embedded Angle in Soil Nailing

(1) Issues

In Japan, it is generally accepted that grouted nails should be placed at an angle downward from the horizontal plane (5–10° according to empirical methods for cut-side reinforcement, and within the range of 10°–30° as specified in the Manual for *Design and Construction of Slope Reinforcement Method* (JGS, 2011)). On the other hand, overseas, the range of grouted nail insertion angles may vary by country, and differences in the approach to determining effective insertion angles present an issue.

(2) Explanation

With regard to the insertion angle of grouted nails in reinforced soil, thorough analysis is necessary from the perspective of their effectiveness, taking into account the geotechnical properties of the slope. Regarding the relationship between insertion angle and reinforcement effectiveness, the experimental results of Jewell (1980) for a uniform strain field, shown in Figure 8.2.8, are widely recognized; these results indicate that reinforcement is effective when the angle of the reinforcing element relative to the slip surface falls within the range of -10° to 90°. In particular, the reinforcement effect is maximized at an angle $\theta = 30°$ relative to the slip surface.

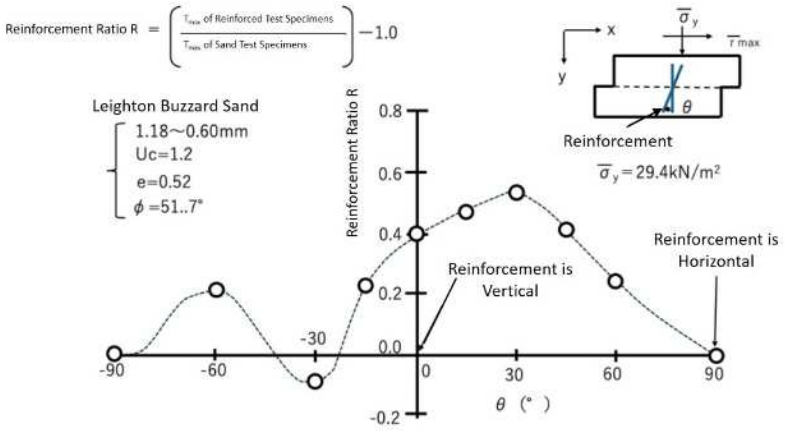


Figure 8.2.8 Results of Jewell's experiments verifying the reinforcement effect based on the insertion angle of the reinforcement (NEXCO, 2024)

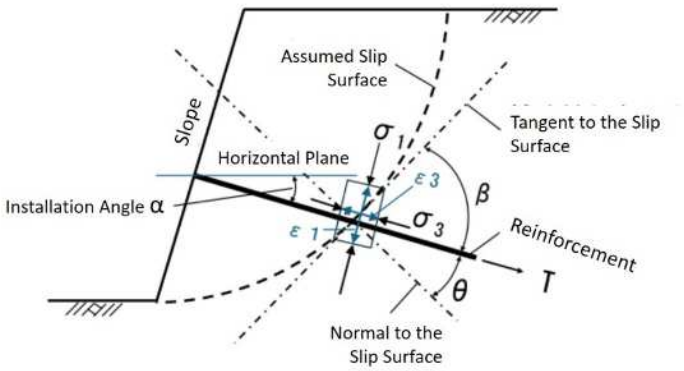


Figure 8.2.9: Insertion Angle of Soil Nail and Reinforcement Effect (JGS, 2011)

This indicates that the reinforcement effect is greatest when the soil nail is inserted in the direction in which the soil stretches the most (the direction of the maximum principal strain ε_3) (see Figure 8.2.9). Furthermore, assuming

that the direction of the maximum principal strain (the ε3 direction) is at an angle of 45° + φ/2 relative to the slip surface, some studies (Tateyama et al., 1993) suggest that there is no significant difference in the reinforcement effect as long as the angle β relative to the slip surface falls within ±15° of the ε3 direction. According to this, for soil with φ = 30°, the range where β is between 45° and 75° corresponds to a range with a high reinforcement effect. In cut slopes with gradients of 60° to 80°, when soil nail is installed perpendicular to the slope face, the angle α relative to the horizontal plane ranges from 10° to 30°, meaning the reinforcement is generally inserted within the region where tensile reinforcement effectiveness is high. Thus, the insertion angle of the soil nail is evaluated from a geotechnical engineering perspective based on tensile reinforcement effectiveness in a uniform strain field.

On the other hand, a field survey conducted by the NEXCO Research Institute (formerly the Japan Highway Public Corporation Research Institute) (NEXCO, 2007a,2024a) reported that 74% of piles were “inserted perpendicular to the slope,” and that “perpendicular to the slope” combined with “horizontal” accounted for 95% of all cases. From the perspective of the reinforcement effects mentioned earlier, this can generally be considered an appropriate insertion angle. In practice, however, when dealing with steep cut slopes, vertical installation is often preferred due to construction considerations such as the quality of the grout (including bleeding) and the connectivity with the slope facing when the reinforcement is installed at an angle other than perpendicular to the slope.

Furthermore, when reinforcing gently sloping slopes, if tensile reinforcement is prioritized, the installation angle of the soil nail must be determined by considering the slope facing structure, the connection with the slope facing, and the workability of the grout material, such as bleeding. Alternatively, the angle of the soil nail must be determined by considering the combined effects of bending and tensile forces.

Figures 8.2.10 to 8.2.13 present items concerning the angle of soil nails as described in the UK Transport Research Laboratory (TRL) research report TRL-20020-TRL537 (TRL, 2002). The soil nail angle is shown to be effective between 30° and 50° upward from horizontal. Regarding the tensile force effect of the soil nail, inserting it upward relative to the slip surface maximizes the tensile reinforcement effect to 100% and minimizes the required soil nail length (TRL, 2002).

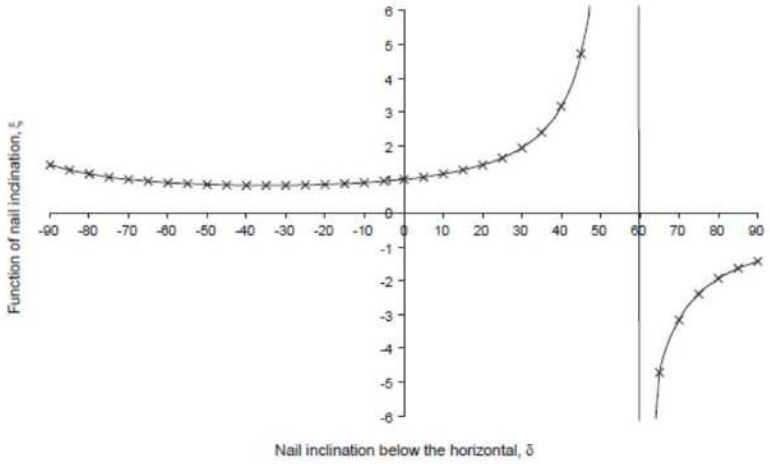


Figure 8.2.10 Change in Tensile Resistance with Nailing Installation Angle (TRL, 2002)

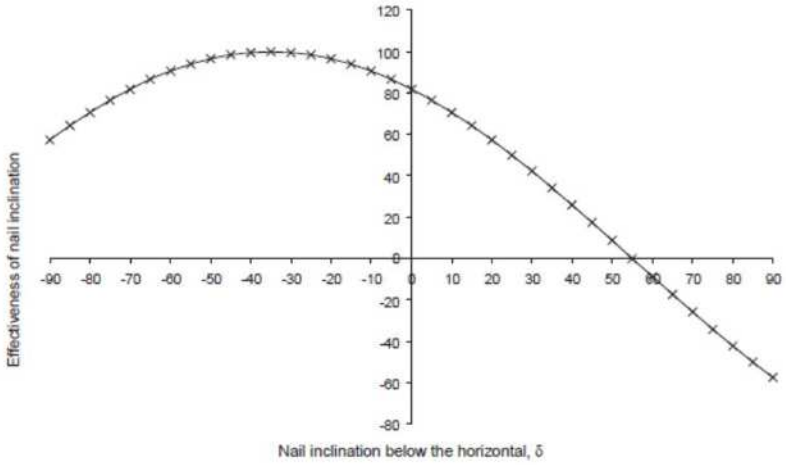


Figure 8.2.11 Effect of Nailing Installation Angle (TRL, 2002)

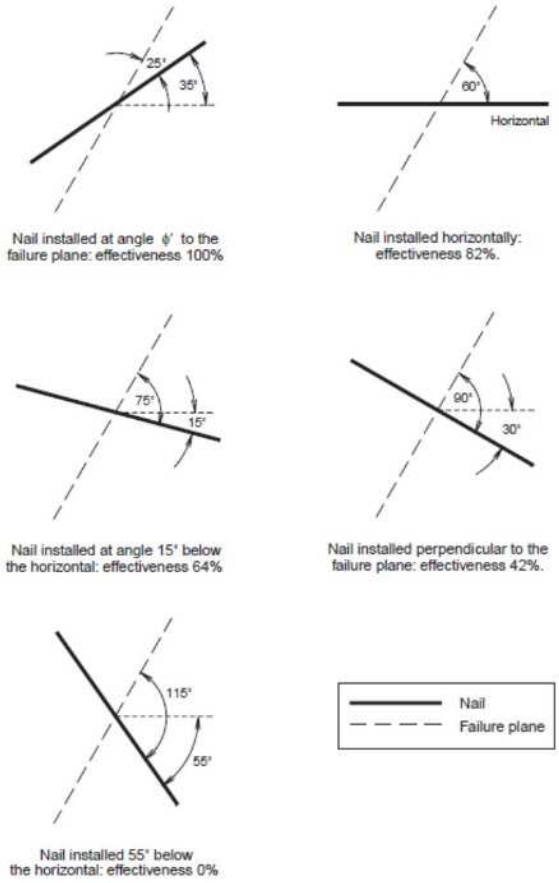


Figure 8.2.12 Theoretical Tensile Efficiency of Nailing Varying with Slope Angle Difference Relative to the Slip Surface (TRL, 2002)

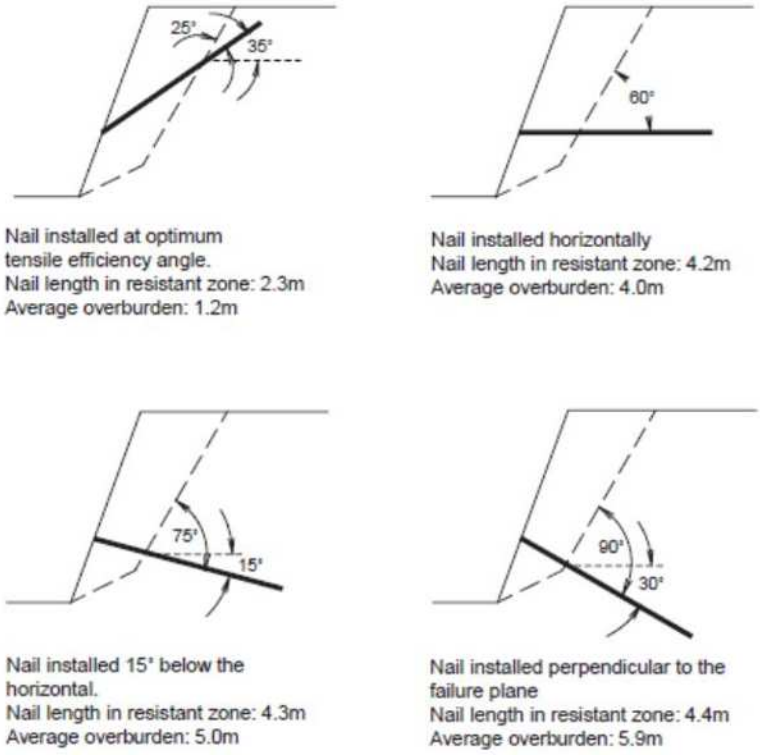


Figure 8.2.13 Nails installed at different efficiency levels—corresponding overlying loads and lengths in the resistance zone (TRL, 2002)

(3) Actions to Take

Since the approach to determining effective soil nail insertion angles is considered to differ by target country, it is necessary to investigate the target country's standard guidelines and respond in accordance with their approach. If no approach exists, explain the Japanese approach and proceed using the Japanese insertion angle. When using an upward insertion angle, as in British standards, and the soil nail length is long, grout injection may become problematic. When determining the insertion angle, the constructability of such grout injection must also be considered.

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8. 2. 4 Limits on Grouted Nail Length for Soil Nailing

(1) Issues

In Japan, the grouted nail length for nailing methods used as slope failure countermeasures is typically adopted within the range of 2m to 5m (JGS, 2011). In contrast, overseas nailing methods tend to set the design slip surface relatively deep based on ground conditions, with numerous cases employing reinforcement lengths exceeding 10 m, indicating differing restrictions on grouted nail length.

(2) Explanation

The length of grouted nails for soil nailing is calculated through design calculations based on factors such as the position, spacing, and installation angle of the grouted nail, the diameter of the grouted nail, and the bond strength of the soil. However, when the volume of the collapse is large and the collapse depth is deep, sufficient reinforcement effectiveness cannot be expected. Considering factors such as the construction capacity of the machinery used for soil nailing, a realistic maximum length is set.

Japanese road standards (NEXCO, 2007) and standards for countermeasures against steep slope failures (JSA, 2019) provide guidelines for the applicable slope failure depth of soil nailing, as shown in Figure 8.2.14. Furthermore, as shown in Figure 8.2.15, statistics indicate that 90% of landslides in Japan are shallow, with a depth of 3 meters or less (Momma et al., 1999). Furthermore, for slopes with relatively deep slip surfaces, ground anchors and pile works are recommended. Considering economic and construction feasibility, the grouted nail length for soil nailing is generally set at 2 to 5 meters.

On the other hand, Railway Standards in Japan (JRCC, 2001, RTRI, 2007) specifies maximum lengths based on the diameter of the grouted nails: up to 5m for small-diameter grouted nails (φ50-100mm or less), 7m for medium-diameter grouted nails (approximately φ100-300mm), and 10m for large-diameter grouted nails (φ300-500mm).

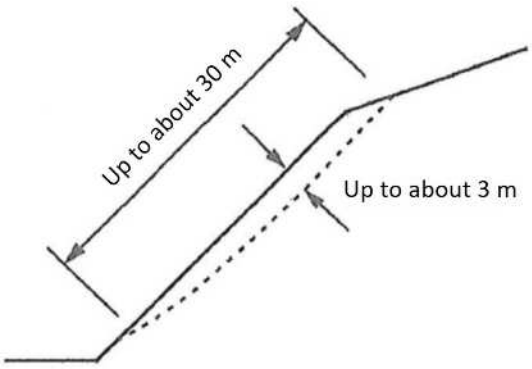


Figure 8.2.14: Guidelines for Applicable Slope Scale of Soil Nailing (JSA, 2019)

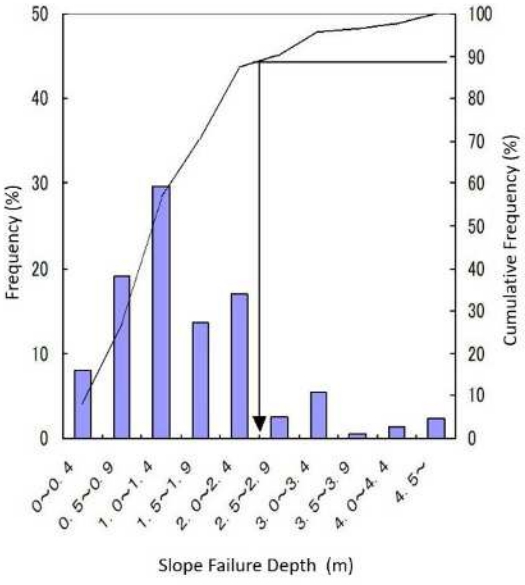


Figure 8.2.15 Slope Failure Depth

(3) Actions to Take

In countries lacking statistical data on failure depth and lacking technologies such as ground anchors or pile

works capable of addressing deep slides, when soil nailing is necessary, it is considered unnecessary to specify the soil nail length. The soil nail length should be determined based on the grouted nail diameter, considering economic feasibility and constructability.

If necessary, soil nails significantly longer than 5m may be used. However, for a slope failure with such deep slip surface, it must be recognized that the restraint effect of soil nailing—which passively generate restraint force— might be more limited and uncertain than that of ground anchors, which forcibly apply restraint force.

In the countries with fragile ground conditions and frequent triggers (such as torrential rain and earthquakes), similar to Japan, long soil nails may fail to prevent collapse. The length of soil nails must be determined considering the ground conditions and climate of the target country.

References

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JSA (2019): *New Design and Examples of Slope Failure Prevention Works: Technical Guidelines for Steep Slope Failure Prevention Works*, Japan SABO Association (JSA), PP.263.

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Momma, K., Chida, Y. and Ebihara, K. (1999): Actual Conditions of Cliff Collapse Disasters, Public Works Research Institute (PWRI), *Technical Note of PWRI*, No.3651, PP.81-123.

8. 2. 5 International Differences in Grouted Nail Spacing

(1) Issues

For soil nailing, grouted nail spacing is considered appropriate at approximately 1 bar per 2 m², based on past construction practices, with a maximum spacing of 1.5 m. However, when using rigid slope facing, spacing may be increased up to 2 m.

Meanwhile, overseas, there are cases with maximum reinforcement spacings of up to 2.5 m (Indian standard) or 3.0 m (Rockscience SLIDE2 Guidelines, Rockscience, 2021), presenting the issues of differing grouted nail spacing restrictions.

(2) Explanation

The spacing of grouted nail is determined to satisfy the conventional two-dimensional safety factor. However, when the spacing in the transverse direction is large, there is a concern of partial collapse in the grouted nail gaps (see Figure 8.2.19). Conversely, when the spacing between them is too small, there is a risk that the expected resistance per grouted nail, similar to the group effect of piles, cannot be achieved in the design. For these reasons, Japanese standards (e.g., JGS, 2011) provide the following empirical values for grouted nail spacing.

Minimum spacing: 1.0 m

Maximum spacing: Based on past construction practices, a density of approximately 1 bar per 2 m² is considered appropriate, with a maximum spacing of 1.5 m. However, when using rigid slope facings, the spacing may be increased up to 2 m (1 bar per 4 m²).

Railway Standards (JRCC,2001; RTRI,2007) show an empirical relationship between the effective diameter of grouted nail and the maximum spacing as a guideline in Figure 8.2.16. According to this figure, the maximum spacing for the nailing method (grouted nail effective diameter approximately 50 mm) is 1.5 m. This represents the maximum grouted nail spacing guideline based on substantial experience with soil nailing in the railway sector, demonstrating the validity of the 1.5 m spacing.

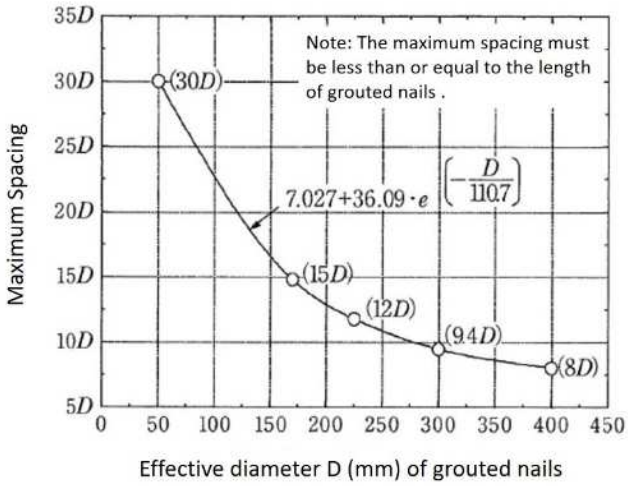


Figure 8.2.16 Relationship between Effective Diameter of Grouted Nail and Maximum Spacing (Guideline) (JGS, 2011)

Furthermore, as shown in Figure 8.2.17, while the grid arrangement equal vertical and horizontal spacings, the staggered arrangement has different vertical and horizontal spacings. Consequently, the placement method for grouted nails is not standardized in practice. When applying this to overseas projects, it is necessary to consider the ground conditions of the target country and to examine the spacing of the grouted nails, taking into account partial collapse (hollowing out) in the gaps between grouted nails and the cluster effect (a phenomenon where the reinforcement capacity per grouted nail is reduced due to the influence of adjacent grouted nails when the spacing is narrow: Figure 8.2.18).

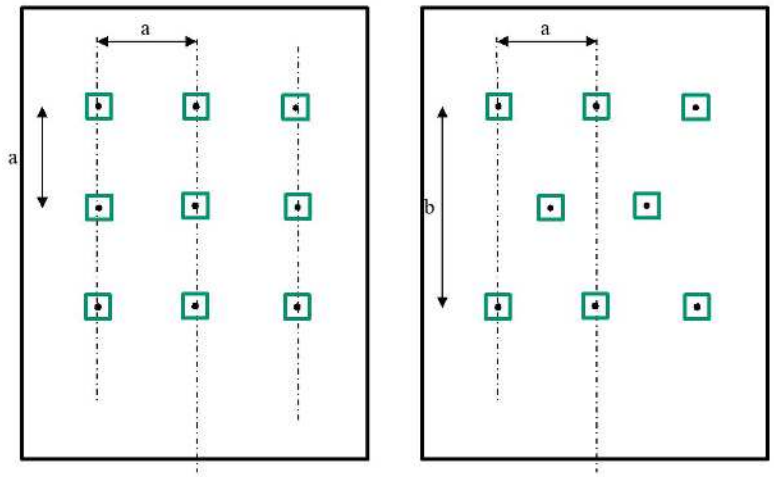


Figure 8.2.17 Lattice arrangement (left) and staggered arrangement (right)

(3) Actions to Take

The grouted nail density should be approximately 1 bar per 2 to 4 m². Set the reinforcement spacing considering the group effect (Figure 8.2.18) and partial collapse between grouted nails (Figure 8.2.19).

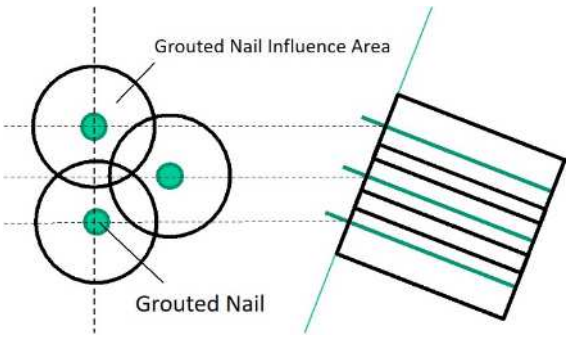


Figure 8.2.18 Group Effect

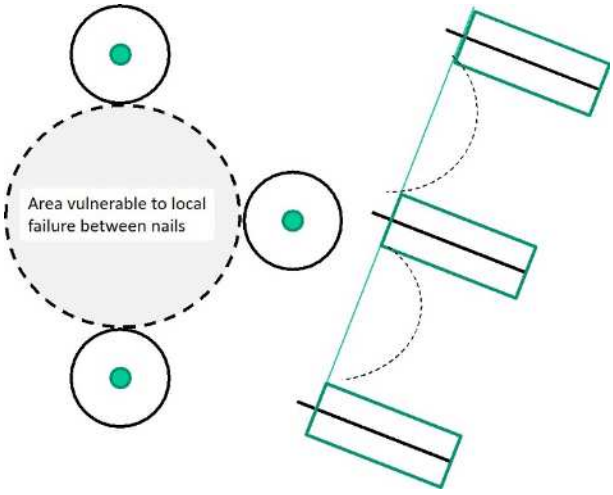


Figure 8.2.19 Local Failure Between Grouted Nails

References

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RTRI (2007): *Design Standards for Railway Structures and Commentary: Earth Structures*, Railway Technical Research Institute (RTRI).

8. 2. 6 Applicability of Bond Strength Used in Japanese Design Conditions

(1) Issues

Japan specifies a guideline (standard value) for the bond strength τ between grout and ground. However, a key issue arises when attempting to directly adopt this Japanese standard value in overseas locations with differing ground conditions. Additionally, a question arises regarding whether to adopt the same safety factor as Japan when calculating the design pull-out resistance.

(2) Explanation and Actions to Take

1) Bond Strength

As a guideline (standard value) for the bond strength between the grouting material and the ground in soil nailing, Table 8.2.5 values are shown in the *Design and Construction Guidelines for Cut Soil Reinforcement Works* (NEXCO, 2007), while Table 8.2.6 values are shown in the *Manual for Design and Construction of Slope Reinforcement Works* (JGS, 2011). Both manuals use the same values for rocks, but they adopt different values for soil (sand and gravel, sand). The values in NEXCO (2007) are based on ground anchor standards, adopting 80% of the lower limit value for each ground classification. The values in JGS (2011) incorporate the latest test data for sand and gravel, setting values equivalent to the lower limit value for each ground classification.

Both adopt the same ground classification as ground anchors, with only one standard value listed per ground classification. Differences in strata strength within a single ground classification are not reflected. The range in standard values for bond strength in Japanese ground anchor works stems from variations in strength even within the same soft rock. The fact that this is not reflected in the standard values for soil nailing is problematic. This further underscore why Japanese standard values cannot be directly applied overseas. Standard values for bond strength are intended for rough estimates of the quantity of countermeasures during preliminary design stages. In practice, prior pull-out tests, as required for ground anchor works, are necessary.

Table 8.2.5 Standard Values for Bond Strength in Japan (1) (NEXCO, 2007)

Type of Ground			Bond Strength (MN/m²)
Rock	Hard Rock*		1.20
	Soft Rock*		0.80
	Weathered Rock		0.48
	very stiff clay		0.48
Gravel	N value	10	0.08
		20	0.14
		30	0.20
		40	0.28
		50	0.36
Sand	N value	10	0.08
		20	0.14
		30	0.18
		40	0.23
		50	0.24
Cohesive soil			0.8×c (c is cohesion)

A classification unique to Japan based on earthwork workability

Table 8.2.6 Standard Values for Bond Strength in Japan (2) (JGS, 2011)

Type of Ground			Bond Strength (KN/m²)
Rock	Hard Rock*		1200
	Soft Rock*		800
	Weathered Rock		480
	very stiff clay		480
Gravel	N value	10	100
		20	130
		30	180
		40	280
		50	450
Sand	N value	10	80
		20	100
		30	150
		40	200
		50	300
Cohesive soil			0.8×c (c is cohesion)

A classification unique to Japan based on earthwork workability

Incidentally, in Europe and the United States, the principle is to obtain bond strength through field tests. For example, the UK's Transport Research Laboratory (TRL, 2002) explicitly states that a safety factor of 1.3 should be applied to values obtained from pull-off tests.

As described later, the standard values for bond strength in soil nailing specified by the U.S. Federal Highway Administration (FHWA, 2015) and the Transportation Research Board (TRB, 2011) employ a rock classification system entirely different from Japan's. For example, even bedrock is categorized by rock type, and standard values for soil are provided corresponding to detailed soil classifications.

(a) U.S. Case Study

The soil-grout interface bond strength values introduced in the U.S. Federal Highway Administration (FHWA) *Soil Nails Walls Reference Manual* (FHWA, 2015) are shown in Tables 8.2.7 to 8.2.9. The interface friction stress varies depending on the drilling method and soil conditions. While direct comparison is difficult since soil hardness is not specified, under the Rotary Drilled conditions in these tables, values are roughly equivalent to those for sandy soils with N=20 under Japanese standards.

表 8.2.7 Estimated Bond Strength for Soil Nails in Coarse-Grained Soils (Modified after Elias and Juran 1991) （FHWA,2015）

Drill-Hole Drilling Method	Soil Type	Bond Strength, q_u (psi)[2]
Rotary Drilled	Sand/gravel	15 - 26
Rotary Drilled	Silty sand	15 - 22
Rotary Drilled	Silt	9 - 11
Rotary Drilled	Piedmont residual	6 - 17
Rotary Drilled	Fine Colluvium	11 - 22
Driven Casing	Sand/gravel w/low overburden [1]	28 - 35
Driven Casing	Sand/gravel w/high overburden [1]	41 - 62
Driven Casing	Dense Moraine	55 - 70
Driven Casing	Colluvium	15 - 26
Augered	Silty sand fill	3 - 6
Augered	Silty fine sand	8 - 13
Augered	Silty clayey sand	9 - 20

Note: (1) Low and high overburden are defined as effective overburden pressure being, respectively, less than and greater than 1.5 tsf. (2)1psi=6.895kPa

表 8.2.8 Estimated Bond Strength for Soil Nails in Fine-Grained Soils (Modified after Elias and Juran 1991)（FHWA,2015）

Drill-Hole Drilling Method	Soil Type	Bond Strength, q_u (psi)(1)
Rotary Drilled	Silty clay	5 - 7
Driven Casing	Clayey silt	13 - 20
Augered	Loess	4 - 11
Augered	Soft clay	3 – 4
Augered	Stiff clay	6 – 9
Augered	Stiff clayey silt	6 – 15
Augered	Calcareous sandy clay	13 - 20

Note: (1) 1psi=6.895kPa

表 8.2.9 Estimated Bond Strength for Soil Nails in Rock – Drilling Method: Rotary Drilled (Modified after Elias and Juran 1991)（FHWA,2015）

Rock Type	Bond Strength, q_u (psi)(1)
Marl/limestone	44 – 58
Phyllite	15 – 44
Chalk	73 - 87
Soft dolomite	58 – 87
Fissured dolomite	87 – 145
Weathered sandstone	29 – 44
Weathered shale	15 – 22
Weathered schist	15 – 25
Basalt	73 – 87
Slate/Hard shale	44 - 58

Note: (1) 1psi=6.895kPa

TRB (2011) proposes Equation (3) for determining the bond strength force using in-hole horizontal loading tests. The coefficients *a* and *b* in this equation are specified for each soil classification shown in Table 8.2.10.

$$q_u = a\left(p_L\right)^b \qquad \cdots(3)$$

Here, q_u: Bond Strength, p_L: Maximum value of the in-hole horizontal load test, *a*, *b*: Soil classification coefficients shown in Table 8.2.10

Figure 8.2.20 shows the relationship between the *N*-value (lower horizontal axis), the borehole horizontal load limit (upper horizontal axis), and the bond strength (vertical axis) for major soil conditions (sand, gravel, cohesive soil, weathered rock). The thick solid line represents the regression curve underlying Equation (3), with its confidence interval indicated by the dashed line.

表 8.2.10 Parameters a and b for equation (3), correlation between q_u and p_L.(TRB, 2011)

Material type	*a*		*b*
	English units (1)	SI units (2)	
Sand	6.90	119	0.390
Gravel	5.87	122	0.469
Clays	5.89	120	0.461
Weathered Rock	6.33	177	0.595

Notes: (1) Enter p_L in tsf to obtain q_u in psi., (2) Enter p_L in MPa to obtain q_u in kPa.

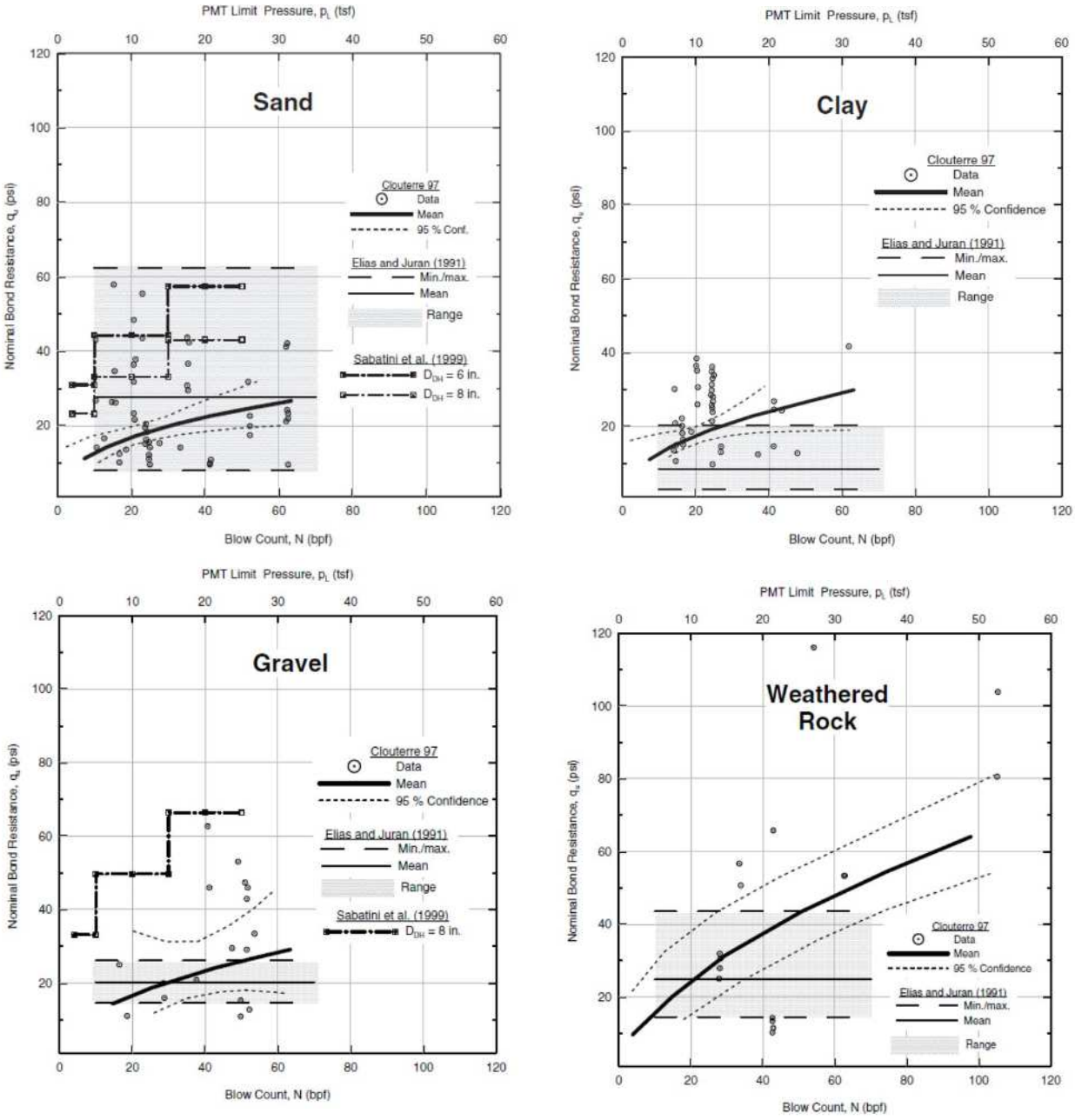


Figure 8.2.20 Relationship between N-value, borehole loading test limit value pL, and bond strength qu in major soil types (TRB 2011)

Table 8.2.11 shows the bond strength obtained from drilling tests conducted experimentally by the FHWA and the International Association of Foundation Drilling (ADSC) on self-drilling grouted nail(pressure-injected). (The values for pressure-injected micropiles show results from tests conducted under similar soil conditions for comparison.) These tests demonstrate that the bond strength of soil nails differs depending on whether grout is injected under pressure or without pressure. The bond strength is greater when injected under pressure than without pressure. However, in soils with a high proportion of fine-grained material (SM), bond strength is obtained even without pressure.

Table 8.2.11 Differences in Bond Strength by Injection Method (FHWA.2015)

Original: Table 10.1: Nominal Bond Strength of Soil Nails in Granular Soils (after Cadden et al. 2010)

Soil Type	Gravity-Grouted Solid Bar Soil Nail (psi)	Hollow Bar Soil Nail (psi)	Pressure-grouted Micropiles (Sabatini et al. 2005) (psi)
Silty Sand (SM)	12.8-17.1	18.7-24.4	Sand (some silt) 10-27.5
Poorly Graded Sand (SP)	13.2	71.2-79.6	Sand (some Gravel) 17.5-52
SP with Gravel	50.5-82.2	-	Sand (some Gravel) 17.5-52
Poorly Graded Gravel (GP) with Sand	53.2-64.5	62.4-156.3	Gravel (some Sand) 17.5-52

Note: (1) 1psi=6.895kPa, (2) Bond strength values from tests are calculated using the nominal drill bit diameter.

(b) Case Study: India

The *Code of Practice for Reinforcement of Rock Slopes with Plane Wedge Failure, 1997* (Bureau of Indian Standards (BIS), 2012) is the standard for soil reinforcement and ground anchors for rock slopes in India.

The values for rock bond strength in India are shown in Table 8.2.12. The RMR values in the table represent evaluation points based on the rock mass classification method using the “Rock Mass Rating (RMR) method” proposed by Bieniawski (1976). Each value is shown within a range that encompasses the rock bond strength values for each Japanese rock mass classification (Tables 8.2.5 and 8.2.6).

表 8.2.12 Safe Bond Strength for Different Rock Conditions

Rock type	Safe Bond Strength(τ_f/F)
Very poor to poor rock (RMR=0 to 40) condition	0.35-0.70 N/mm^2
Fair to good rock (RMR=41 to 80) condition	0.70-1.05 N/mm^2
Very good rock (RMR=81 to 100) condition	1.05-1.40 N/mm^2

2) Safety Factor for Pull-Out

The safety factor for pull-out between the ground and grout material is set in Japan as shown in Table 8.2.13 and is slightly smaller than that for ground anchors.

Table 8.2.13 Safety Factors for Pull-Out of Grouted Nails (JGS, 2011)

Load Condition	Safety Factor
Non-seismic conditions	2.0
Temporary conditions	1.5
Seismic conditions	1.25
Construction conditions	1.25 or 1.5

Overseas examples are shown below.

(a) Hong Kong Case Study

This section describes the *Guide to Soil Nail Design and Construction* (CEDD, 2008) compiled by Hong Kong's Civil Engineering and Development Department (CEDD). Table 8.2.14 is a table published in CEDD (2008). The FT in the table is the safety factor for the allowable tensile strength TT of the grouted nail. FGR is the safety factor for the allowable pull-out resistance TGR, which depends on the bond length between the mortar and the reinforcement. FSG is the safety factor for the allowable pull-out resistance TSG in the active zone under combined tensile, shear, and bending actions. TSG is calculated based on cohesion and internal friction angle, not the bond strength of the target soil.

Table 8.2.14 specifies different safety factors for the pull-out resistance TSG of grout in soil, depending on the soil type.

- Weathered granite or volcanic rock: Safety factor 1.5
- For continuous loads or temporary loads acting on soils other than weathered granite or volcanic rock: Safety factor 2.0

表 8.2.14 Recommended Minimum Factor of Safety against Internal Failure of a Soil Nail(CEDD, 2008)

Mode of Internal Failure	Minimum Factor of Safety
Tensile failure of soil-nail reinforcement	$F_T = 1.5$
Pullout failure at soil-grout interface	$F_{SG} = 1.5$ (Note 1) $F_{SG} = 2.0$ (Note 2)
Pullout failure at grout-reinforcement interface	$F_{GR} = 2.0$

Notes: (1) For soil nails carrying transient loads and bonded in weathered granite or volcanic rocks. (2) For soil nails carrying sustained loads or for soil nails carrying transient loads and bonded in soils other than weathered granite or volcanic rocks.

(b) UK Case Study

This explains the content of the Transport Research Laboratory (TRL) research report (TRL, 2002). The standards used when considering reinforced soil for slope stabilization in the UK include HA68 (*Design methods for the reinforcement of highway slopes by reinforced soil and nailing technique*), BS8006:1995 (*Code of practice for strengthened/reinforced soil and other fills*), and BS8081:1989 (*Code of practice for ground anchorages*). BS 8006:1995 primarily covers embankment reinforcement with retaining walls, while BS 8081:1989 addresses

ground anchors. The former includes content on rock reinforcement in addition to standard embankment reinforcement, which is also relevant to reinforced soil. The latter covers not only conventional ground anchors but also anchors without prestressing, and this section is also relevant to soil reinforcement.

Table 8.2.15 compares these three standards. The red-marked row corresponds to the Pull-out capacity section. For HA68, only the pull-out coefficient based on testing and residual strength is listed, with no notation for the safety factor. BS 8081:1989, which specifies the safety factor for ground anchors, sets a safety factor of 3.0 relative to the ultimate design load.

表 8.2.15 Comparison of design methods and their partial factors (TRL, 2002)

	HA 68	BS 8006:1995	BS 8081:1989
Design approach	Limit state	Limit state	Limit state
Analysis	Limiting equilibrium	Limiting equilibrium	Limiting equilibrium
Shape of failure surface	Two-part wedge (applies to <70° slopes)	Two-part wedge or log-spiral (applies between 10° and 70° slopes)	Single and multiple wedge for steep slopes, circular for shallow slopes <27°
Representative soil parameters	Minimum conceivable	Cautious estimate/worst credible	No specific guidance
Water regime	Conservative values	No specific guidance	No specific guidance
Material factor on tanφ′	Varies: 1.0 - 1.5 on residual, critical state or peak	1.0	n/a
Material factor on c′	1.0 - 1.5 on peak strength (5 kN/m2 max.)	1.6	n/a
Material factor on Cu	Not used	Not used	Adhesion factor = 0.3 to 0.35 on Cu
Minimum surcharge	Not given	Not given	Not given
Load factors		ULS SLS	
Vertical soil loads	1.0	1.5 1.0	Overall FS = 1.5
Vertical dead loads		1.2 1.0	
Vertical surcharge loads	1.0	1.3 1.0	Overall FS = 1.5
Non-vertical soil loads	As vertical	As vertical	As vertical
Non-vertical surcharge loads	As vertical	As vertical	As vertical
Pull-out capacity	Interface sliding factor based on tests or residual strength	1.3	FS = 3 on ultimate load to derive design load
Base sliding	Depends on interface sliding factor	1.2	Not covered explicitly

(c) U.S. Case Study

This explains the content of the Federal Highway Administration (FHWA)'s *Soil Nails Walls Reference Manual* (FHWA, 2015). Table 8.2.16 shows the safety factors for key design items of soil reinforcement method (soil reinforcement retaining structures) at the Federal Highway Administration. The safety factor for pull-out, highlighted in red, is 2.0 under non-seismic conditions and 1.5 under seismic conditions, which is the same as in Japan.

表 8.2.16 Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method (1)（FHWA, 2015）

Limit State	Condition	Symbol	Minimum Recomm. Factors of Safety, Static Loads	Minimum Recomm. Factors of Safety, Seismic Loads
Overall	Overall Stability	FS_{OS}	1.5(2)	1.1(6)
Overall	Short Term Condition, Excavation	FS_{OS}	1.25 – 1.33(3)	NA
Overall	Basal Heave	FS_{BH}	2.0(4), 2.5(5)	2.3(5)
Strength – Geotechnical	Pullout Resistance	FS_{PO}	2.0	1.5
Strength – Geotechnical	Lateral Sliding	FS_{LS}	1.5	1.1
Strength – Structural	Tendon Tensile Strength (Grades 60 and 75)	FS_{T}	1.8	1.35
Strength – Structural	Tendon Tensile Strength (Grades 95 and 150)	FS_{T}	2.0	1.50
Strength – Structural	Facing Flexural	FS_{FF}	1.5	1.1
Strength – Structural	Facing Punching Shear	FS_{FP}	1.5	1.1
Strength – Structural	Headed Stud Tensile (A307 Bolt)	FS_{FH}	2.0	1.5
Strength – Structural	Headed Stud Tensile (A325 Bolt)	FS_{FH}	1.7	1.3

Notes: (1) The limit state and symbol nomenclature differ from that presented in the previous version of this manual. Many of these changes reflect the move toward using LRFD terminology as presented in AASHTO (2014).
(2) For non-critical, permanent structures, some Owners may accept a design for static loads and long-term conditions with FSOS = 1.35 when uncertainty is considered to be limited due to the availability of sufficient geotechnical information and successful local experience on soil nailing.
(3) This range of safety factors for global stability corresponds to the case of temporary excavation lifts that are unsupported for up to 2 days before nails are installed. The larger value may be applied to critical structures or when more uncertainty exists regarding soil conditions.
(4) This factor of safety for basal heave is applicable to permanent walls for short-term conditions.
(5) This factor of safety for basal heave is applicable to permanent walls for long-term conditions.
(6) The minimum FSOS for seismic overall stability should be 1.0, when horizontal seismic coefficients are used and these were derived from estimated, allowable seismic deformations.

(d) Case Study: India

The *Code of Practice for Reinforcement of Rock Slopes with Plane Wedge Failure, 1997* (Bureau of Indian Standards (BIS), 2012) recommends a safety factor *F* of 3 to 5 for pull-out resistance.

(3) Actions to Take

1) Response Regarding Bond Strength

Organize the standards and guidelines of the relevant country. If no standard values are specified, consider the country's standard values by referring to standard values corresponding to N values or soil types, such as those in the US TRB (2011). The principle is that the relevant country should also determine the bond strength value by conducting an ultimate pull-out test prior to design.

2) Response Regarding Pull-Out Safety Factor

If the target country has regulations for the safety factor, comply with those values. If no regulations exist, the Japanese safety factor shall be adopted for the time being. However, it is desirable to eventually establish the country's own original safety factor, referencing pull-out test results from that country and similar geological cases from other nations.

References

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8. 2. 7 Applicability of Japan's Design Safety Factors

(1) Issues

In Japan, design safety factors of 1.2 (under non-seismic conditions) and 1.0 (under seismic conditions) are generally used for soil nailing. However, design safety factors for slope protection measures vary by country.

(2) Explanation

Table 8.2.17 shows examples of design safety factors for earth structures in representative fields within Japan. Although the first edition of *Guidelines for Highway Earthworks*, published in Showa 31 (1956) (JRA, 1956), contains no mention of the design safety factor, the Showa 42 edition (JRA, 1967) states in the section titled "Measures Against slow-moving Landslides and Collapses" that the minimum safety factor for cut works must be 1.2 or higher. The Showa 47 edition (JRA, 1972) contains a similar statement. From that time onward, a design safety factor of 1.2 became the standard for collapse countermeasures in road standards.

The safety factor for earth structures in railway standards was first proposed in *Guidelines for Design and Construction of Earth Structures (Draft)* (JNR, 1968). For the stability of embankments and cuttings, standard slopes were established based on soil and geological conditions. However, when the soil composition forming the cutting slope face was homogeneous and the cross-sectional shape was simple (e.g., without terraces), the required cohesion was determined using stability charts. At this time, the safety factor was defined as $F_c = C/C_d$ (C: cohesion of the soil, C_d: cohesion required for stability). It was recommended that a safety factor of 1.3 or greater be maintained for both cohesive and sandy soils. Similarly, a safety factor of 1.3 or greater was also required for slow-moving landslide stability.

Following the privatization of the national railways in 1987 and their transition to JR, the stability of embankments was categorized into two phases: during construction and after opening. The safety factor for the former was set at 1.2, while that for the latter was set at 1.4. These values were established in the 1992 revision of *Design Standards for Railway Structures and Commentary: Earth Structures* (RTRI, 1992) established required performance and performance ranks (Okada & Sugiyama, 2008). This background explains the difference in design safety factors between road and railway standards. Note that in Japan, the design safety factor is based on inverse analysis, whereas in Europe and America, it is based on forward analysis.

Table 8.2.17 Examples of Design Safety Factors for Earth Structures (JGS, 2011)

Conditions	Design Safety Factor		
	Road Standard(2)	Railway Standard(3)	
	Circular Slide	Circular Slide	Two-wedge slide (1)
Non-seismic	1.2	1.4	2.0
Temporary	-	1.4	1.5
Seismic	-	1.1	1.25
Construction	1.05 or 1.10	1.2	-

Note: (1): Two-line sliding indicates a collapse pattern due to wedge failure. (2): NEXCO (2007), (3): JRCC (2001) and RTRI (2007)

Table 8.2.18 shows representative design safety factors used overseas. Additionally, Tables 8.2.19 to 8.2.21 present design safety factors from the United States, Hong Kong, and the UK Road Research Laboratory. Overseas, since forward analysis is primarily used, soil testing is necessary. While samples suitable for soil testing can be

collected from the better-quality soil within the target ground, sampling from the weakest soil is difficult.

Table 8.2.18 Examples of Design Safety Factors by Country (Bong, 2014MS)
Original: Table 2.2 Comparison between various references on the Factor of Safety (FOS) requirements

References	FOS Requirements
BS 6031 (in UK)	1.3-1.4 for first time slide 1.2 for slide with pre-existing slip surface
JKR Road Works (in Malaysia)	1.2 for unreinforced slope & embankment on soft ground 1.5 for reinforced slope
Hong Kong Geoguide	1.0-1.4 for new slopes depending on risk categories 1.0-1.2 for existing slope depending on risk categories
NAVFAC DM7.1 (in USA)	1.5 for permanent loading condition 1.15-1.2 for transient load
Britain National Coal Board 1970 (in UK)	1.5/1.35 (peak/residual strength used) for risky slope 1.25/1.15 (peak/residual strength used) for non-risky slope
Canada, Mines Branch 1972	1.5/1.3 (peak/residual strength used) for risky slope 1.3/1.2 (peak/residual strength used) for non-risky slope

(a) U.S. Case Study

This section introduces a case study from *Soil Nails Walls Reference Manual* (FHWA, 2015). Table 8.2.19 shows the design safety factors published in FHWA (2015). Design safety factors of 1.5 under non-seismic conditions, 1.1 under seismic conditions, and 1.25 to 1.3 for short-term (temporary) and construction service are indicated.

Table 8.2.19 Design Safety Factors for Soil Nail Walls Applied in FHWA (2015) (1)
Original: Table 5.1 Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method

Limit State	Condition	Symbol	Minimum Recomm. Factors of Safety, Static Loads	Minimum Recomm. Factors of Safety, Seismic Loads
Overall	Overall Stability	FS_{OS}	1.5(2)	1.1(4)
Overall	Short Term Condition, Excavation	FS_{OS}	1.25-1.33(3)	NA

Notes: (1) The limit state and symbol nomenclature differ from that presented in the previous version of this manual. Many of these changes reflect the move toward using LRFD terminology as presented in AASHTO (2014).
(2) For non-critical, permanent structures, some Owners may accept a design for static loads and long-term conditions with FSOS = 1.35 when uncertainty is considered to be limited due to the availability of sufficient geotechnical information and successful local experience on soil nailing.
(3) This range of safety factors for global stability corresponds to the case of temporary excavation lifts that are unsupported for up to 2 days before nails are installed. The larger value may be applied to critical structures or when more uncertainty exists regarding soil conditions.
(4) The minimum FSOS for seismic overall stability should be 1.0, when horizontal seismic coefficients are used and these were derived from estimated, allowable seismic deformations.

(b) Hong Kong Case Study

The Geotechnical Engineering Office of Hong Kong's Civil Engineering and Development Department (CEDD) presents design safety factors for applying soil nails to cut slopes in Table 8.2.20 (CEDD, 2008). These values are shown as design safety factors against a 10-year probability rainfall, with the maximum design safety factor being 1.4 across all categories.

Table 8.2.20 Design Safety Factors for Soil Nails Applied to Cut Slopes (CEDD, 2008)

Economic Consequence	Consequence-to-life		
	Category 1	Category 2	Category 3
Category A	1.4	1.4	1.4
Category B	1.4	1.2	1.2
Category C	1.4	1.2	> 1.0

Notes: (1) In addition to a minimum factor of safety of 1.4 for a ten-year return period rainfall, a slope in the consequence-to-life category 1 should have a factor of safety of at least 1.1 for the predicted worst groundwater conditions.

(2) The factors of safety given in this Table are recommended minimum values. Higher factors of safety might be warranted in particular situations in respect to loss of life and economic loss.

Here, Categories 1, 2, and 3 indicate classifications based on the magnitude of human losses caused by damage to protecttion assets, with Category 1 representing the most significant impact. Categories A, B, and C indicate classifications based on the magnitude of economic losses caused by damage to protection assets, with Category A representing the most significant impact. Examples for each classification are shown in Table 8.2.22 and Table 8.2.23.

表 8.2.21 Typical Examples of Slope Failures in Each Consequence-to-life Category(CEDD, 2008)

Consequence-to-life	Examples
Category 1	1) Failures affecting occupied buildings (e.g., residential, educational, commercial or industrial buildings, bus shelters, railway platforms). 2) Failures affecting buildings storing dangerous goods.
Category 2	1) Failures affecting heavily used open spaces and recreational facilities (e.g., sitting-out areas, playgrounds, car parks). 2) Failures affecting roads with high vehicular or pedestrian traffic density. 3) Failures affecting public waiting areas (e.g., bus stops, petrol stations).
Category 3	1) Failures affecting country parks and lightly used open-air recreational areas. 2) Failures affecting roads with low traffic density. 3) Failures affecting storage compounds (non- dangerous goods).

表 8.2.22 Typical Examples of Slope Failures in Each Economic Consequence Category(CEDD, 2008)

Economic Consequence	Examples
Category A	1) Failures affecting buildings, which could cause excessive structural damage. 2) Failures affecting essential services, which could cause loss of that service for an extended period. 3) Failures affecting rural or urban trunk roads or roads of strategic importance.
Category B	1) Failures affecting essential services, which could cause loss of that service for a short period. 2) Failures affecting rural (A) or primary distributor roads which are not sole accesses.
Category C	1) Failures affecting open-air car parks. 2) Failures affecting rural (B), feeder, district distributor and local distributor roads which are not sole accesses. 3) Failures affecting country parks.

(c) UK Case Study

Table 8.2.23 compares the key design factors specified in the three main UK guidelines (HA68, BS8006:1995, BS8081:1989) for soil nails. This table is taken from a document on soil nails published by the Transport Research Laboratory (TRL, 2002). HA68, the design standard for Soil Nails on motorways, specifies partial safety factors for earth mass weight and imposed loads but does not define a design safety factor for overall sliding. BS8006:1995, the standard for reinforced embankments with retaining walls, specifies partial safety factors for ULS (Ultimate Limit State) and SLS (Serviceability Limit State). The standard BS8081:1989 for ground anchors specifies a design safety factor for the entire slope.

Table 8.2.23 Comparison of Design Methods and Their Sub-Factors (TRL, 2002, Excerpt)

	HA 68	BS 8006:1995		BS 8081:1989
Design approach	Limit state	Limit state		Limit state
Analysis	Limiting equilibrium	Limiting equilibrium		Limiting equilibrium
Shape of failure surface	Two-part wedge (applies to <70° slopes)	Two-part wedge or log-spiral (applies between 10° and 70° slopes)		Single and multiple wedge for steep slopes, circular for shallow slopes <27°
Representative soil parameters	Minimum conceivable	Cautious estimate/ worst credible		No specific guidance
Water regime	Conservative values	No specific guidance		No specific guidance
Material factor on tanφ′	Varies: 1.0 - 1.5 on residual, critical state or peak	1.0		n/a
Material factor on c′	1.0 - 1.5 on peak strength (5 kN/m^2 max.)	1.6		n/a
Material factor on C_u	Not used	Not used		Adhesion factor = 0.3 to 0.35 on C_u
Load factors		ULS	SLS (1)	
Vertical soil loads	1.0	1.5	1.0	Overall FS = 1.5
Vertical dead loads		1.2	1.0	
Vertical surcharge loads	1.0	1.3	1.0	Overall FS = 1.5
Pull-out capacity	Interface sliding factor based on tests or residual strength	1.3		FS = 3 on ultimate load to derive design load
Base sliding	Depends on interface sliding factor	1.2		Not covered explicitly

Notes: (1) ULS: ultimate limit states, SLS: serviceability limit states.

Additionally, British Standard BS 6031:1981 (Technical Specification for Earth Structures) (UK, 1981) recommends a safety factor of 1.5 against slope instability for permanent structures over the long term.

(3) Actions to Take

Since design safety factors vary by country, it is desirable to design after first clarifying the relevant country's standards and guidelines.

If the country has regulations for slope countermeasure design safety factors, comply with those values. If no regulations exist, adopt the design safety factor used in Japan or similar countries for the time being.

References

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JRA (1956): *Guidelines for Highway Earthwork*, Japan Road Association (JRA)

JRA (1967): *Guidelines for Highway Earthwork*, Japan Road Association (JRA)

JRA (1972): *Guidelines for Slope Protection and Slope Stability Engineering*, Japan Road Association (JRA)

8. 2. 8 Implementation of Various Field Surveys to Obtain Design Constants and Inadequate Environmental Conditions

(1) Issues

In Japan, various field tests such as standard penetration tests and simplified penetration tests required for classifying ground conditions are conducted when designing soil nailing. Ultimate pull-out tests (suitability tests) are also performed to obtain bond strength. Numerous technical manuals concerning these tests have been published, and equipment such as simplified penetration test machines and jacks for conducting the tests is readily available. Overseas, the environment for conducting these test surveys is often not adequately established.

(2) Explanation

While many companies overseas possess the capability to conduct standard penetration tests, simplified penetration tests are often not performed. Furthermore, in countries where ground anchor construction and soil nailing are not common practices, the necessary equipment and materials for conducting ultimate pull-out tests using jacks and similar devices are unavailable.

If such survey and testing methods are not established in the country in question, an explanation of these methods must also be included in the design calculation documents for soil nailing. Furthermore, it is necessary to explain the need to obtain the necessary ground information for design to ensure the engineer's understanding, and to introduce the survey methods and equipment used in Japan, providing guidance on their handling and the utilization of survey results.

<Reference Notes: Example of Simplified Dynamic Penetration Test Method>

The figure shows an overview of the simplified dynamic penetration tester, and the figure shows the test conditions. The tester measures the number of blows (N_d value) required for a specified cone to penetrate 10 cm by freely dropping a 5 kg weight from a height of 50 cm. The N value is then calculated using Equation (1).

Based on the relationship $N = N_d/(1 \text{ to } 3)$, Equation (1) is generally used.

$$N = N_d / 1.5 \quad \cdots (1)$$

Furthermore, using Equation (2), the internal friction angle φ of the soil can be determined from the N-value.

$$\phi = 4.8 \log N + 21 \quad (N > 5) \quad \cdots (2)$$

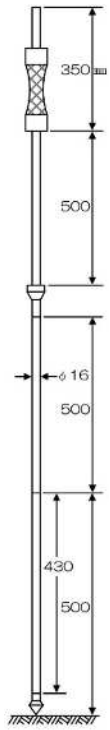


Figure 8.2.21 Simple Penetration Tester



Figure 8.2.22 Simple Penetration Test in Progress

References

JRA (2012): *Guidelines for Highway Earthwork: Retaining Wall*, Japan Road Association (JRA), PP.64.

8. 2. 9 Differences in Corrosion Resistance of Steel Nails

(1) Issues

In Japan, it is standard practice to apply corrosion protection, such as hot-dip galvanizing, to the reinforcing steel used as core material in the design of soil nailing. In contrast, overseas, materials without corrosion protection are frequently used.

(2) Explanation

Soil nailing is a method that passively provides restraint force through deformation. While they are based on the premise of injecting grout around the steel nail, sufficient corrosion resistance is required against water ingress through cracks in the grout caused by deformation. Furthermore, since the top of the steel nail is exposed at the ground surface, high corrosion resistance is required. Therefore, a minimum plating thickness of 550 g/m² (JIS H 8641, ISO 1461:2009) must be applied along the entire length. Furthermore, during design, a corrosion allowance (Highway Standard (NEXCO, 2007): 0.5 mm; Railway Standard (JRCC, 2001, RTRI, 2007): 1 mm).

(a) Hong Kong Case Study

Table 8.2.24 shows the corrosion protection methods adopted by Hong Kong's CEDD (2008). It specifies a hot-dip galvanizing coating weight of 610 g/m² or more, which is greater than the standard values generally prescribed in Japan.

Table 8.2.24 Corrosion Protection Standards in CEDD (2008)

<table>
<tr><th rowspan="2">Design Life</th><th colspan="4">Soil Aggressivity Classification</th></tr>
<tr><th>Highly aggressive</th><th>Aggressive</th><th>Mildly aggressive</th><th>Non-aggressive</th></tr>
<tr><td>Up to 120 years</td><td colspan="2">Class 1</td><td colspan="2">Class 2</td></tr>
<tr><td>Up to 2 years (Note 2)</td><td colspan="4">Class 3</td></tr>
<tr><td colspan="5">Corrosion Protection Measures:
Class 1 - Hot-dip galvanising with a minimum zinc coating of 610 g/m^2 and corrugated plastic sheathing
Class 2 - Hot-dip galvanising with a minimum zinc coating of 610 g/m^2 and 2 mm sacrificial thickness on the radius of the steel reinforcement
Class 3 - Hot-dip galvanising with a minimum zinc coating of 610 g/m^2</td></tr>
<tr><td colspan="5">Notes: (1) For “potentially aggressive” sites without soil aggressivity assessment, Class 1 corrosion protection measures should be provided to soil nails with a design life more than 2 years.
(2) Soil aggressivity assessment is not required for soil nails with a design life up to 2 years.</td></tr>
</table>

(b) Case Study: Philippines

Item-522 (DPWH, 2019), one of the Department Orders of the Philippine Department of Public Works and Highways (DPWH), specifies the requirements for the use of soil nailing in road slope stabilization. This standard was revised in 2019 based on the business activities of the ITEM522 Revision Committee, composed of Japanese and European companies, compared to the previous ITEM522 (2014 edition).

Among the slope stabilization techniques adopted by the Philippine Department of Public Works and Highways (DPWH), Soil Nailing is classified as an Active System (source control) (DPWH, 2019). Active Systems are further categorized into Non-Mesh Systems and Mesh Systems (Table 8.2.25).

Table 8.2.25 Types of Soil Nailing Adopted by the Philippine DPWH (DPWH, 2019)

Kind	Description
1. Non-mesh system	The slope is stabilized using steel plates and anchors at specified spacing, diameter and length reinforced with wire rope. If necessary, the use of steel wire mesh can be allowed as a supplementary member for surface erosion control or surface rock pinning. If required, the use of erosion control materials (Items 512 and 622) and/or hydroseeding (Item 520) can be allowed.
2. Mesh system	The slope is stabilized using mesh systems reinforced with steel plates and anchor bolts/soil nails at specified spacing. Additional layer of erosion control materials (Items 512 and 622) and/or hydroseeding (Item 520) can be allowed.

Table 8.2.26 shows the corrosion protection methods for soil nailing in the Non-mesh system adopted by the Philippine DPWH. The steel bars used in the Non-mesh system are hot-dip galvanized at 550 g/m², the same as in Japan. This table also incorporates JIS standards. Meanwhile, the specifications for the Mesh system are shown in three separate tables; however, these only list the type of corrosion protection and do not specify the quantity.

Table 8.2.26 Corrosion Protection for Soil Nailing Materials in Non-Mesh Systems (DPWH, 2019)

Component	Specification	Test Method
Steel Plate	Welded Structural Steel Size: (Thickness) 6 mm × (Height) 95mm × (Width) 680 mm Area: 0.22 m^2 Yield strength: 245 MPa (Min.)	ASTM A36M or any equivalent Standards such as JIS G3101:SS400
	Coating: Galvanized Zinc coating: 550 g/m^2	Coating: ASTM A653M / ASTM or any equivalent Standards such as JIS H 8641 and JIS H 0401/ ISO 1459-1973
Wire Rope	Diameter: 8 mm Breaking Load: 33.8 kN (Min.)	ASTM A1023M or any equivalent Standards such as JIS G 3525
	Zinc-alloy coating: 90%ZN & 10%AI (100 g/m^2)	Coating: ASTM A90M or any equivalent Standards such as JIS H 0401
	Accelerated weathering (salt spray) test: 80% of tensile strength shall be retained after 3,500 hours	Corrosion resistance: ASTM B117 or any equivalent Standards such as JIS Z 2371
Anchor Bolt	Diameter of self-drilling type: 28.5 mm Yield strength: 450 MPa (Min.) Diameter of insert-after-drilling type: 19 - 32 mm Yield strength: 345 MPa (Min.) Zinc Coating: 550 g/m^2 or equivalent	ASTM A615M I ASTM A29M or any equivalent Standards such as JIS G 3112 I JIS G 4051 Coating: ASTM A653M/ ASTM A90M or any equivalent Standards such as JIS H 8641/JIS H 0401
Wire Mesh (optional)	Diameter: 2mm (Min)	ASTM A 1023/ASTM A931 (except wire tensile strength) or any equivalent standard
Grout	If necessary, grouting requirements shall conform to the applicable provisions of Item 513, Permanent Ground Anchors.	

(4) U.S. Case Study

The corrosion protection described in the U.S. Federal Highway Administration (FHWA) Technical Manual on Soil Nails (FHWA, 2015) recommends using high-density polyethylene (HDPE) as specified in AASHTO M 252 or corrugated PVC pipe as specified in ASTM D1784. Furthermore, when applying hot-dip galvanizing, AASHTO 2014 and ASTM A123 specify a minimum of 2.0 oz/ft² (610 g/m²) or a thickness of 3.4 mil (86.36 μm).

The following is an excerpt regarding corrosion protection performance from FHWA (2015).

3.2.5 Corrosion Protection Elements (FHWA,2015)

In addition to the cement grout, which provides both physical and chemical protection to the tendons, other devices are used to provide additional corrosion protection, as necessary. The solid bar tendon can be protected by encapsulation in a sheath of corrugated high-density polyethylene (HDPE) conforming to AASHTO M252, or of corrugated PVC tube conforming to ASTM D1784, Class 13464-B. After the bar is installed in the sheathing, the annulus between the sheathing and the tendon is filled with grout prior to leaving the manufacturer. Ultimately, once the encapsulated bar is placed in the drill hole, the annulus between the sheath and the drill hole is grouted using tremie methods as described previously.

Fusion-bonded epoxy coating can be applied to solid bars. The epoxy coating is a dielectric material that impedes the flow of electric currents that favor corrosion. Fusion-bonded epoxy coating must meet the requirements of ASTM A775. There are several different types and thicknesses of epoxy coating for bars. The color of the coatings is green, gray or purple. The green epoxy coating is flexible and suitable for steel reinforcement that will be bent in different shapes. The gray and purple epoxy coatings are less flexible than the green epoxy coating but have greater chemical resistance. The purple epoxy coating is better suited for marine or harsh environments.

Testing has been conducted on resistance to abrasion and performance of the coating by various manufacturers; however, there is not sufficient data available in the technical literature to provide a direct correlation between the numerical results of resistance to abrasion tests and the actual performance of the protective coating in the field.
Solid and hollow tendons and their hardware can also be hot-dip galvanized. In the case of HBSNs, however, because drilling is performed with the hollow bars themselves, it is likely that these coatings, as well as epoxy coatings, are damaged during installation (Cadden et al. 2010; Samtani and Nowatzki 2010).

Appendix E: Soil Nail Wall Procedural Specifications (FHWA,2015)

1.3.2 Soil Nails
1.3.2.a Soil Nail Solid Bar (Tendon)

Tendons shall meet the requirements of AASHTO M31/ASTM A615 for Grade 60 or 75 steel bars (Grade 420 or 520 in SI Units), and ASTM A722 for Grade 150 steel (Grade 1035 in SI Units). Tendons shall be continuous without splices or welds, new, straight, undamaged, bare, epoxy coated, galvanized, or encapsulated as shown on the Contract Drawings. The length of the threaded portion of the bar at the wall anchorage shall be as needed to allow proper attachment of the bearing plate and nut. If threads are cut into a tendon, the contractor shall verify that the bar meets the minimum capacity required at the threaded section.

1.3.2.b Bar Couplers

Bar couplers shall develop the full nominal tensile capacity of the tendon as certified by the manufacturer.

1.3.2.c Fusion-Bonded Epoxy Coating

Fusion-bonded epoxy coating shall meet the requirements of ASTM A775 and have a minimum thickness of 12 mils (12 thousandths of an inch) up to a maximum of 17 mils as applied electrostatically. Bend test requirements are waived. The coating at the wall anchorage end of epoxy-coated bars may be omitted over the length provided for threading the nut against the bearing plate.

1.3.2.d Zinc Coating

Zinc galvanized coating shall meet the requirements of Article 11.10.6.4.2a (AASHTO 2014 or current edition) and have a minimum of 2.0 oz/ft^2 or 3.4 mil thickness applied in accordance with ASTM A123 for bars and structural steel shapes, and ASTM A153 for nuts, plates, and other hardware. [ASTM A767/A767M can also be used as a reference.]

1.3.2.e Encapsulation
Bar encapsulation shall be a minimum 0.04-in. thick, corrugated, HDPE tube conforming to AASHTO M252, or corrugated PVC tube conforming to ASTM D1784, Class 13464 B filled with neat cement grout.

(3) Actions to Take

While countries like the Philippines and Hong Kong pay attention to corrosion resistance, most countries have low awareness of the corrosion resistance of steel nails. As a result, inexpensive materials that ignore corrosion resistance are often used. It is necessary to explain that applying specifications with longer service life can ultimately reduce lifecycle costs.

Engineers in these countries must take countermeasures during design: if high corrosion-resistant materials are unavailable locally, they should source them from Japan or similar countries, or design using local materials while factoring in corrosion allowance.

References

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8. 2. 10 Overseas Reinforcing Steel Specifications Differing from Japan

(1) Issues

In Japan, reinforcing steel with material grades SD295 to SD490 and diameters D19 to D38, used as core material for soil nailing, is readily available. However, overseas, fully threaded deformed bars and fully threaded hollow bars with high strength and large outer diameters are difficult to obtain.

(2) Explanation

To maintain the long-term stability of slopes reinforced with soil nailing, the steel nail must be a material that possesses the required tensile properties. Therefore, in Japan, a design standard tensile strength of 295 N/mm² or higher is required. Materials manufactured in factories with thorough quality control and process management, meeting specified standards, quality, shape, and dimensions, are used. Overseas, however, it is often difficult to obtain high-strength materials like SD295 or larger-diameter deformed bars, or the materials may be substandard in quality, shape, or dimensions.

Table 8.2.27 lists the types and characteristics of steel nails used in Japan, while Table 8.2.28 shows their material properties and strength ratings.

Table 8.2.27: Types and Characteristics of Steel Nails (Reinforcing Bars) (JGS, 2011)

Steel Nail Type	Material	Characteristics
Full-threaded deformed steel bar※	Deformed steel bar JIS standard	・Exhibits a plateau in the stress-strain curve. ・Threaded along the entire length.
Deformed steel bar	Same as above	・Exhibits a plateau in the stress-strain curve. ・Cross-sectional defects are present in the threaded section.
Twisted steel bar	High-strength steel produced through cold working and chemical composition adjustment	・Does not exhibit a distinct yield point. ・Slightly lower elongation. ・Cross-sectional defects exist in the threaded section.
Fully thread hollow bar	High-strength steel with rolled threads on a steel tube	・Does not exhibit a distinct yield point. ・Slightly lower elongation. ・Threads extend along the entire length.

*Fully threaded steel bars are classified as threaded bars in JIS M 2506 and are included in deformed bars.

Table 8.2.28: Material Properties and Strength of Major Steel Nails (Steel Bars) (JGS, 2011)

Type	Material	Nominal Diameter	Threaded Section				Material Section		
			Diameter (mm)	Cross-sectional Area (mm^2)	Yield strength (kN)	Ultimate strength (kN)	Cross-sectional Area (mm^2)	Yield strength (kN)	Ultimate strength (kN)
Fully threaded deformed steel bar	SD295	D19	19.1	287	84.7	126	Same as the threaded section		
		D22	22.2	387	114	170			
		D25	25.4	507	150	223			
		D29	28.6	642	189	282			
		D32	31.8	794	234	349			
		D35	34.9	957	282	421			
		D38	38.1	1140	336	502			
	SD345	D19	M20	287	99.0	141			
		D22	M22	387	134	190			
		D25	M24	507	175	248			
		D29	M27	642	221	315			
		D32	M30	794	274	389			
	SD395	D35	M22	957	330	469			
		D38	M22	1140	393	559			
Deformed steel bar	SD295	D19		238	70.2	105	287	84.7	126
		D22		303	89.4	133	387	114	170
		D25		353	104	155	507	150	223
		D29		459	135	202	642	189	282
		D32		561	165	247	794	234	349

(3) Actions to Take

Examine design feasibility using materials available in the country. If design is impossible due to material specification issues, consider importing materials from Japan or other countries.

References

JGS (2011): *Manual for Design and Construction of Slope Reinforcement Method*, The Japanese Geotechnical Society (JGS), PP.5-17.

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