Techniques for reinforcement or renovation of levees

Final

May 2026

PREPARED BY
Technical Committee on
Geotechnical Aspects of Dikes and Levees
(TC201)

Authors Part I and editors Part II:
Meindert Van (Deltares, The Netherlands)
Yida Tao (Deltares, The Netherlands)
Remy Tourment (INRAE, France)
Alessandro Tarantino (University of Strathclyde, United Kingdom)
Hirotoshi Mori (Yamaguchi University, Japan)
Cor Zwanenburg (Deltares, The Netherlands)

Citation: Van, M.A., Tao, P.Y., Tourment, R., Tarantino, A., Mori, H. and Zwanenburg, C. Techniques for reinforcement or renovation of levees. International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) – Technical Committee TC201 'Geotechnical aspects of dikes and levees', May 2026. Download https://doi.org/10.53243/R0008

May 2026

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Preface

Dikes and levees are among the most important pieces of infrastructure in many countries. They protect lives, homes, industry, agriculture, and critical services. When they perform well, they are almost invisible. When they do not, the consequences can be sudden and severe. In many regions, these structures are also ageing, while the demands placed on them are increasing (Hughes, Tisherman, Warren May & Miro, 2024, and Karagiannis, Turksezer, Alfieri, Feyen & Krausmann, 2017).

Reinforcement and renovation are therefore not optional activities. They are a key part of responsible flood risk management. Higher water levels, longer high-water durations, and more frequent extreme events can expose weaknesses that were acceptable in the past but are no longer tolerable today. At the same time, the value of the protected area often grows over time, and society's expectations regarding safety, reliability, and transparency continue to rise. Investing in reinforcement is, in practice, investing in continuity: keeping communities safe, keeping economies functioning, and giving emergency services and decision makers a system they can trust.

A practical starting point for reinforcement is understanding how and why levees fail. In its previous report on failure paths for levees (Van et al., 2022), TC201 described failure mechanisms and how they can combine into sequences of events that ultimately lead to flooding. That work helps us ask the right questions during assessment: what is the dominant mechanism here, what triggers it, and what is the likely progression? The present report builds directly on that foundation by focusing on the next step: which techniques are available to reduce the likelihood of those failure paths, and how can they be selected, designed, and implemented in real projects.

Because levees exist in many different settings, namely rivers, coasts, deltas, estuaries, and mountain valleys, no single country, agency, or guideline covers all situations equally well. International cooperation is essential. Lessons from one flood, one soil condition, or one construction method can often be translated, carefully, into better practice elsewhere. The purpose of TC201 is to support that exchange: to bring together experience from different regions, compare approaches, and make practical knowledge easier to access for engineers, researchers, owners, and authorities.

This publication is written with that goal in mind. Part I provides a structured overview of reinforcement and renovation techniques, organised around key geotechnical failure mechanisms. It also discusses practical considerations that often matter in projects, such as available space, constructability, environmental constraints, and uncertainty about future loading. Part II complements this overview with case descriptions and technique factsheets contributed by TC201 members and their networks. Together, these parts aim to help readers move from problem recognition to an informed shortlist of measures, and from there to a well-argued design and implementation strategy.

We hope that this report supports day-to-day engineering decisions as well as longer-term policy and research discussions. We also hope it encourages continued sharing of data, methods, successes, and failures; because strengthening our dikes and levees is a shared challenge, and the benefits of improved safety extend far beyond national borders.

Dr. Cor Zwanenburg
Chair ISSMGE-TC201

Acknowledgement

This international publication represents a collaborative effort to which members of the International Society of Soil Mechanics and Geotechnical Engineering, Technical committee 201, Geotechnical Aspects of Dikes and Levees and of the International Commission on Large Dams Working Group on Internal Erosion contributed with cases and techniques on the reinforcement or renovation of levees. The effort was coordinated by Meindert Van (Deltares, The Netherlands), Yida Tao (Deltares, The Netherlands), Remy Tourment (INRAE, France), Alessandro Tarantino (University of Strathclyde, United Kingdom), Hirotoshi Mori (Yamaguchi University, Japan) and Cor Zwanenburg (Deltares, The Netherlands). They authored Part I of the report and reviewed and edited the contributions in Part II. Esther Rosenbrand (Deltares) played an important role in Part 1 and in the survey and Lisa van der Linde (Deltares) in giving the report to its current appearance.

Deltares INRAE University of Strathclyde Glasgow YAMAGUCHI UNIVERSITY

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1 Introduction

1.1 Background and Aim

For centuries, levees have been essential for protecting societies from flooding. Understanding of their design and construction has progressively developed over time. Guidelines such as The International Levee Handbook (2013) enable designers to compute failure probabilities and to design levees capable of withstanding prescribed water levels. Mechanisms and failure paths that can lead to damage of the levee and ultimately flooding have been reviewed by the TC201 ‘Dykes and Levees’ (Van et al., 2022).

Processes such as degradation of levee performance, increased hydraulic loading, growth in population and economic value of the hinterland, and improved understanding of failure mechanisms may necessitate levee renovation or reinforcement. The methods used for this purpose depend on the failure mechanisms and failure pathways affecting the performance of the levees. In addition, other considerations may influence the selection of reinforcement techniques. These may relate to uncertainties on the long-term evolution of the levee loading, including sea level rise, land subsidence, and changes in the frequency or intensity of extreme weather events. Environmental concerns can also play a role, including minimising carbon dioxide emissions during construction, and biodiversity and ecological restoration or enhancement. Societal considerations surrounding levees, such as changing attitudes towards acceptable risk and approaches to organizing flood protection, may also play a role. To address reinforcement needs, a range of retrofitting solutions is available, and novel techniques are being developed.

This international report on the renovation and reinforcements of levees was prepared by members of the ISSMGE TC201 ‘Dykes and levees’. The report aims to support practitioners in selecting appropriate methods for their specific challenges and facilitate knowledge exchange among international experts on both traditional and innovative approaches. The report provides an overview of reinforcement and renovation techniques, addressing the main geotechnical failure mechanisms associated with high-water events (e.g. not due to earthquake loading or other non-hydraulic actions). The techniques presented focus on reinforcement and renovation measures, but do not cover repair actions (short-term interventions following major damage) nor periodical maintenance activities .

1.2 Approach

To identify the needs of the international community relevant to this report, a survey was conducted among designers and consultants, agencies responsible for levee operation and maintenance, and researchers. This provided valuable insight into key needs related to levee reinforcement and renovation.

Subsequently, members of the TC 201 contributed information, case studies, and experience with reinforcement techniques used to address these needs. These contributions are presented in Part II of the report and are used to develop an overview of techniques available for specific failure mechanisms. Based on this inventory, recommendations are formulated regarding the selection of appropriate renovation and reinforcement techniques, as well as the identification of potentially promising innovative approaches.

1.3 Report structure

Part I of the report presents an overview of renovation and reinforcement techniques. As terminology used in international practice may vary between countries and among practitioners from different disciplinary backgrounds, the report first introduces key terminology and fundamental concepts. This is followed by a presentation of the requirements identified for this report, as evidenced by the outcomes of the international survey.

The key outcome of Part I is an overview of techniques used to address the main failure mechanisms that drive renovation or reinforcement needs, including references to illustrative examples presented in Part II. An analysis of this framework, including comparisons between countries, is also provided. In addition, Part I offers a brief discussion of potential additional considerations and their influence on a selection of commonly applied techniques, followed by the resulting conclusions and recommendations.

Part II presents an inventory of case examples of reinforcement and renovation techniques contributed by members of the ISSMGE TC201. Each chapter includes a description of a case and/or a technique factsheet, detailing the relevant failure mechanisms and the application of the associated technique. The examples encompass both the reinforcement and renovation of existing levees using established conventional methods, as well as pilot projects and experimental applications involving innovative techniques.

Appendix A presents details of the survey and its results.

Part I: Framework of reinforcement and renovation techniques

Authors Part I:
Core team of Technical Committee on Geotechnical Aspects of Dikes and Levees (TC201)

2 Part I: Framework of reinforcement and renovation techniques

2.1 Terminology and key concepts

Terminology may vary across countries and among practitioners from different backgrounds; therefore, an overview of key terminology and concepts is presented first.

The definitions used in this report are drawn from the ISSMGE TC201 report on ‘Failure paths for levees’ (Van et al., 2022). A failure mechanism is a physical process that can lead to degradation, damage, or collapse of a structure or one of its components. A failure path is defined as the sequence of events by which an initiating event, typically high-water loading, ultimately leads to flooding of the protected area. Although the term *failure path* implies a linear progression of events, multiple mechanisms may occur simultaneously and interact over time. In such cases, the concept of *failure tree*, which indicates branching and interacting pathways, provides a more accurate description.

Structural management of levees refers to the systematic approach used to keep a levee safe, stable, and effective over its lifespan. It combines monitoring, maintenance, and engineering interventions to prevent failure and to adapt to changing conditions such as aging, erosion, or increased flood risk. It includes interventions aimed at restoring a system to its original, as-designed safety performance level, or at improving its performance beyond the original design. These interventions may be implemented either without fundamentally altering the original design intent, or to meet new requirements and address deficiencies not considered in the original design.

Structural management can be broken down into Regular Maintenance and Minor Repairs, Major Repairs and Renovation, and Reinforcement and Retrofitting (Figure 2.1):

i) Regular Maintenance and Minor repairs
Regular maintenance encompasses preventive maintenance and repairs (as mutually exclusive options), and routine activities such as servicing mechanical components and mowing grass-covered revetments, typically performed by the levee management staff or local contractors. Maintenance may involve replacing components of a structure whose service life is shorter than that of the overall structure, or addressing localised areas that have been damaged or are at risk of failure, as defined in *The International Levee Handbook* (2013).
Minor repairs are aimed at restoring the structure based on issues identified during routine inspections, such as filling small cracks or depressions, patching eroded section, sealing minor seepage paths, and repairing small sections of slope protection

ii) Major Repairs and Renovation (also referred to as Rehabilitation)
It is the process of restoring an asset with the objective of returning it to its original, as-designed level of performance after damage has occurred (e.g., due to events causing major seepage or internal erosion issues, large erosion zones or slope failures, or damage of structural components) or its functionality has been reduced due to ageing and deterioration.

iii) Reinforcement (also referred to as Refurbishment or Retrofitting) is the process of improving the safety/and or performance of an asset beyond its original design level. This may include measures to upgrade historical levees for which no original design standard is known. Reinforcement of a function entails adding, upgrading, or enhancing one or more components that contribute to a specific function (i.e. sealing, drainage, filtration, controlled overflow, etc.).

This terminology is consistent with *The International Levee Handbook* (2013) that also distinguishes between returning a levee to its original design performance and improving its performance beyond that level. However, the terminology used in the Handbook differs slightly from that adopted in this report. In the Handbook, Renovation is defined as the process of returning an asset to its original as-designed performance, Rehabilitation refers to restoring an asset for the purpose of returning that asset to design performance, while Reinforcement is defined as the process of improving the performance of a structure, or one of its components, against a specific loading event or degradation mechanism.

This report focuses on techniques of Major Repairs/Renovation and Reinforcement of levees. Regular maintenance and minor repairs are outside the scope of this report.

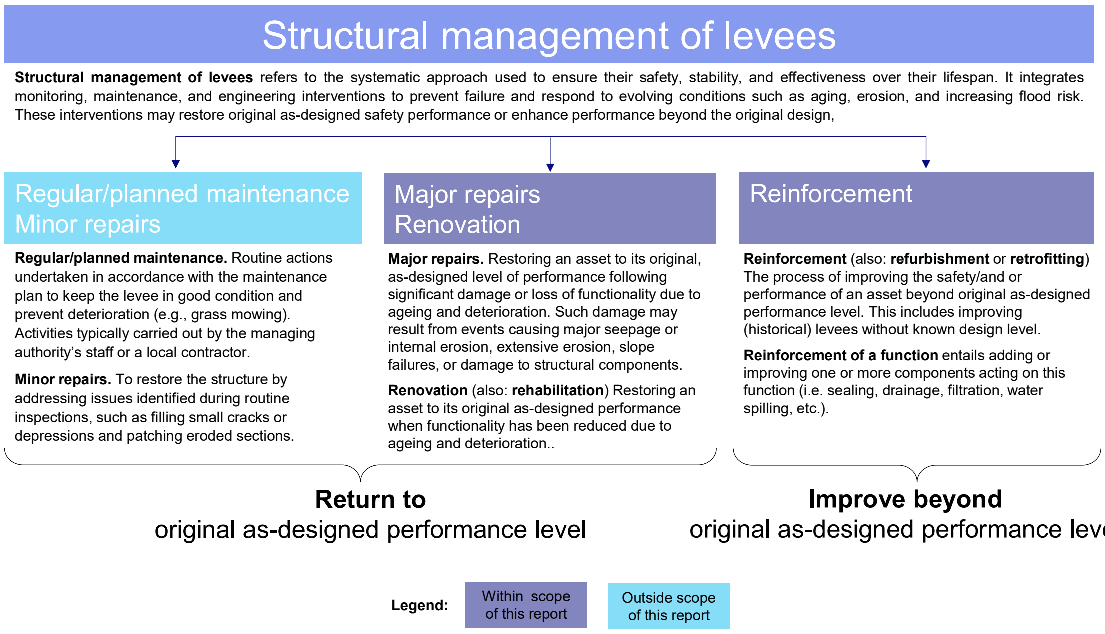

Figure 2.1 Structural management of levees .

2.2 Failure mechanisms and failure paths

Various physical processes, mechanisms, or combinations thereof can affect a levee and lead to failure and subsequent flooding under hydraulic loading.

Considering hydraulic loading – such as by wave loading or elevated water levels – as the initiating event, Figure 2.2 provides an overview of how different failure mechanisms may combine to form failure paths . For further discussion of failure paths, as well as an overview of site-specific characteristics influencing the occurrence of different mechanisms, the reader is referred to the *International Levee Handbook* (CIRIA, French Ministry of Ecology, and USACE, 2013) and the ISSMGE TC201 *Failure Paths* report (Van, Rosenbrand, Tourment, Smith, & Zwanenburg, 2022).

In a given situation, multiple phenomena may occur simultaneously, giving rise to parallel paths, with potential interactions and feedback loops between mechanisms. The thick black lines in Figure 2.2 are used to limit the number of arrows: a phenomenon with an arrow pointing to a black line may lead to the occurrence of one or more of the phenomena shown below that line. For example, an increase in pore-water pressure within the embankment may trigger slope instability. The bidirectional arrows between seepage and increase in pore-water pressure, as well as between internal/external erosion, indicate that these phenomena can interact and mutually influence one another.

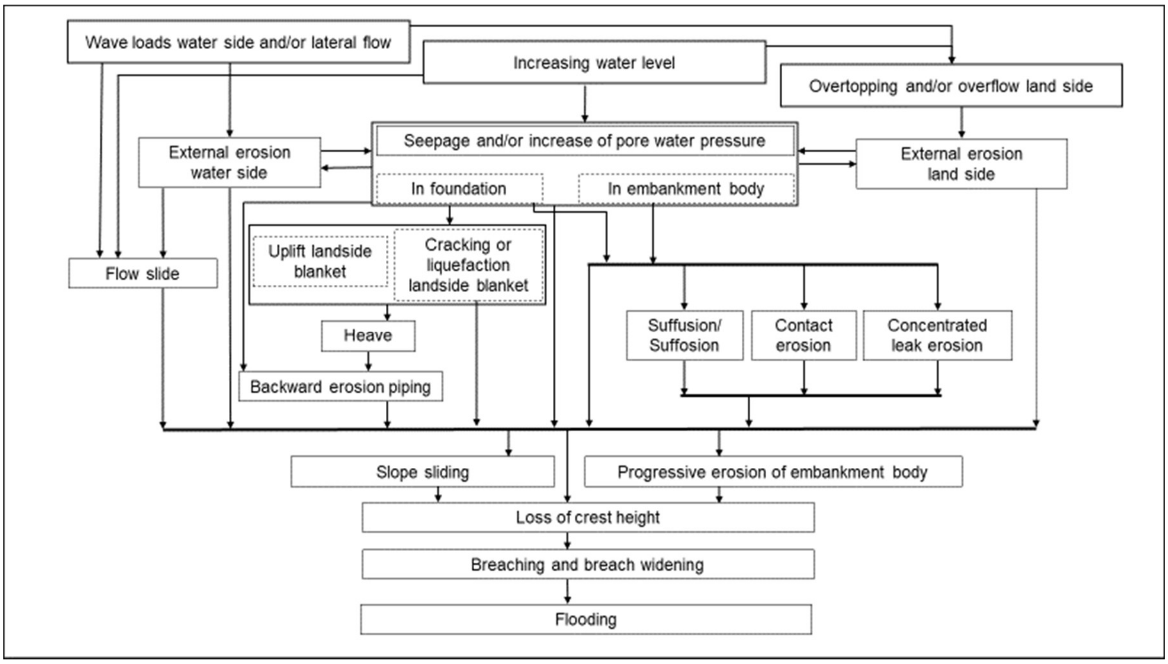

Figure 2.2 The mechanisms are combined into a failure tree to illustrate how factors can combine to initiate a breach (Van, Rosenbrand, Tourment, Smith, & Zwanenburg, 2022).

2.3 International survey on report needs

To capture the needs of the international community, a survey was circulated during the winter of 2023-2024 among the members of the TC201, who subsequently distributed it within their professional and academic networks. Participants were invited to provide input on their needs relating to knowledge on the design and implementation of renovation and reinforcement techniques, as well as their requirements concerning additional considerations and uncertainties in design.

The main results are briefly presented here, while Appendix A contains the survey questions and the corresponding responses.

Participants background

A total of 67 participants responded to the survey. The respondents are involved in renovation and reinforcement activities in a range of different roles, as shown in Figure 2.3, and originate from different continents as shown in Figure 2.4. The majority of respondents are active as consultants or engineers, and many reported experience across multiple roles over the course of their careers.

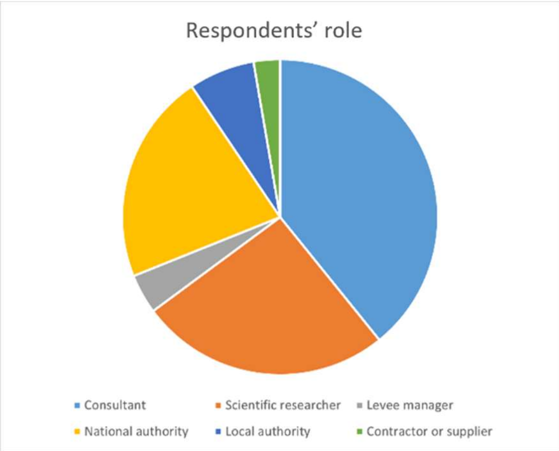

Figure 2.3 Respondents' roles

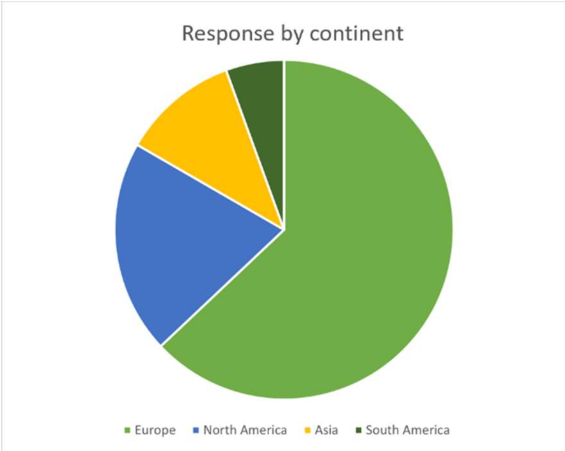

Figure 2.4 Response by continent

Failure mechanisms and environments of concern

The majority of respondents have experience working in riverine or coastal environments. In addition, some respondents reported experience with estuarine levees and levees along mountain streams, as shown in Figure 2.5.

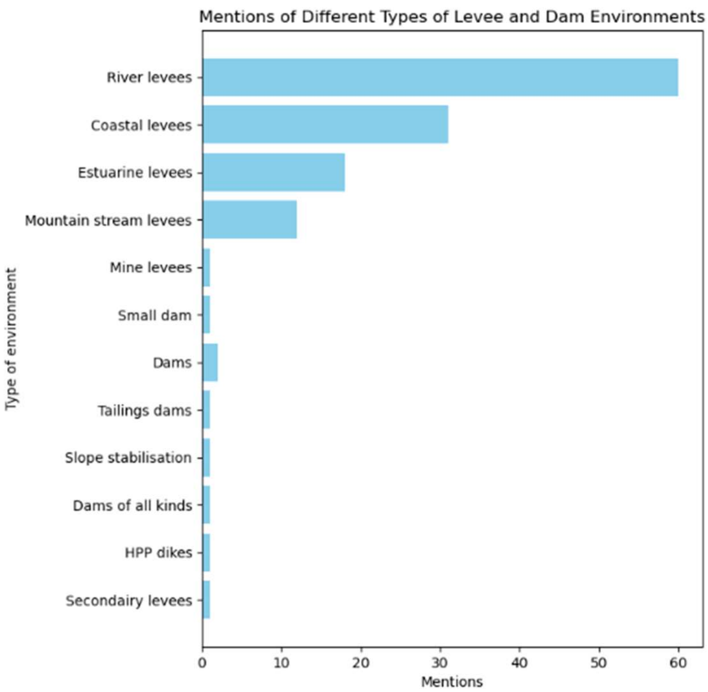

Figure 2.5 Overview of experience by environmental setting

As shown in Figure 2.6, the survey identified three main families of mechanisms:

- internal erosion;
- slope stability (sliding);
- external erosion on the water side slope.

The influence of animal activity and vegetation also represents an important concern for many respondents, as shown in Figure 2.6.

Several respondents indicated that the failure mechanisms of primary concern strongly vary depending on the specific situation and geometry of a levee. They also noted that, for a given levee, multiple mechanisms are often relevant and may act in combination.

Seepage and increase of pore-water pressure were not explicitly addressed in the survey. These are hydraulic conditions that influence failure mechanisms and are implicitly accounted for through failure paths associated with mechanisms such as internal erosion and slope instability. Accordingly, techniques aimed at controlling seepage and pore water pressure are also included within the scope of this report.

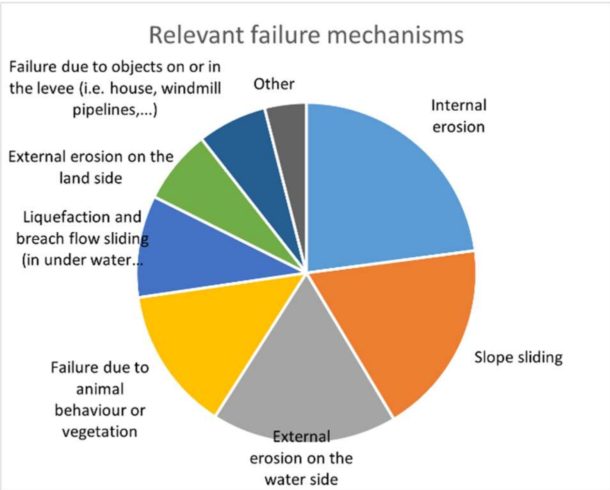

Figure 2.6 Overview of relevant failure mechanisms

The category ‘Other’ included mentions of:

- Failure of dikes reinforced with structural elements
- Seismic conditions and failure of the foundation
- New dimensioning heights
- Lack of maintenance
- Overtopping and inability to operate closures in the levee system

Importance of long-term developments and uncertainties in short- and long-term design

Respondents were asked to assess the extent to which long-term uncertainties influence their design decisions for levee reinforcement and renovation. The responses indicate that increases in flood frequency, intensity, and duration associated with climate change together with environmental considerations, represent the most important concerns when defining design criteria and evaluating long-term uncertainties (see Figure 2.7 and Figure 2.8).

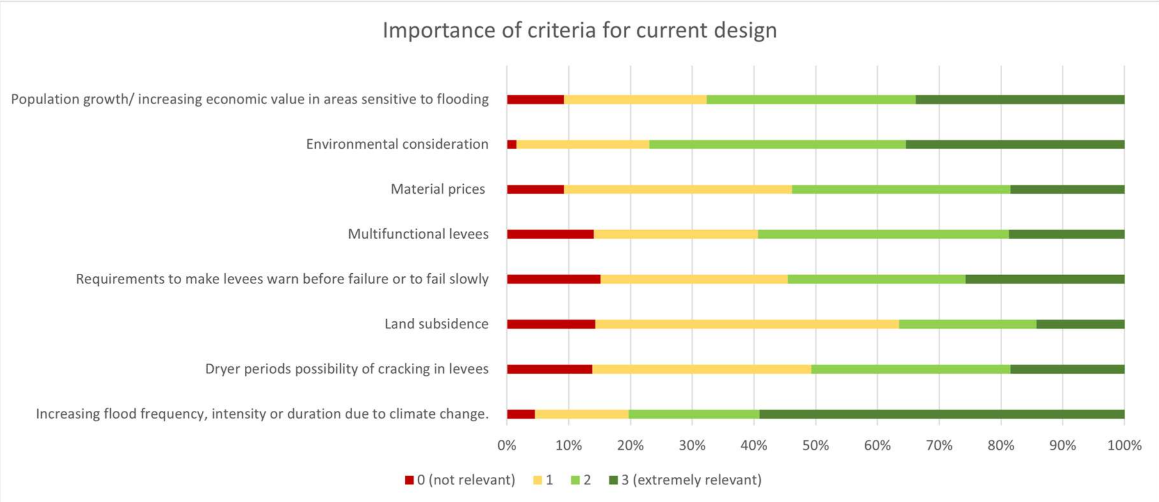

Figure 2.7 Overview of criteria and their importance for current design

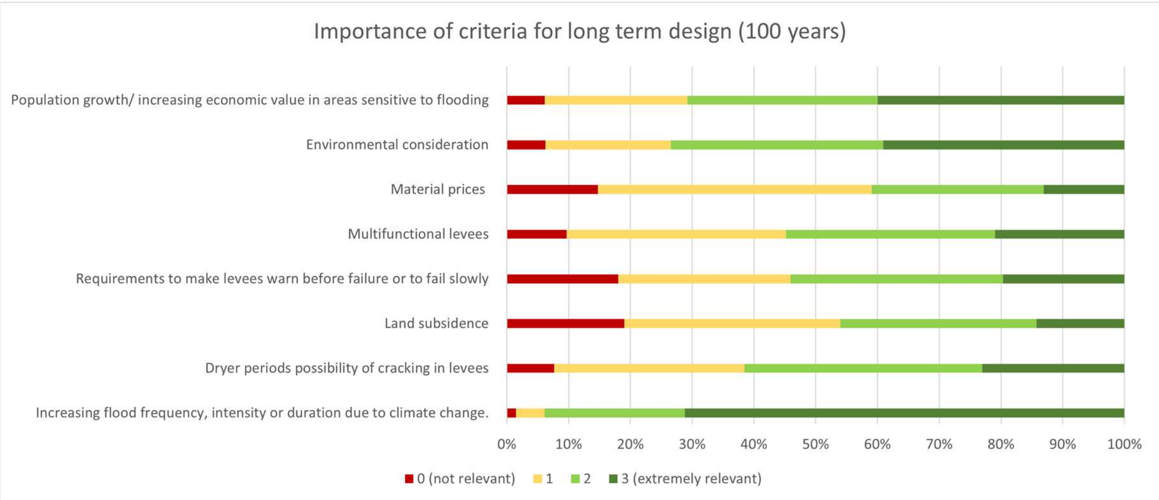

Figure 2.8 Overview of criteria and their importance for long term design (100 years)

Identified needs related to techniques and guidelines

Most respondents find that existing techniques and guidelines can only partially meet their needs. In particular, there is a strong demand for guidance on innovative techniques. Among the techniques of interest, the use of geosynthetics and soil mixing was mentioned repeatedly. Respondents also expressed desire for approaches that enable levees to exhibit more ‘ductile’ or progressive failure behaviour rather than brittle.

2.4 Framework

Members of the ISSMGE TC201 contributed information, case studies, and experience related to levee renovation and reinforcement techniques. These contributions are presented in Part II of the report, which provides an inventory of case examples and technique factsheets. Each contribution includes a description of a case study and/or a technique factsheet, detailing the relevant failure mechanisms and the application of the associated technique.

Based on the case studies and factsheets contributed by experts from different countries, recommendations will be formulated regarding the selection of appropriate renovation and retrofitting techniques, as well as the identification of potentially promising innovative approaches. A comparison between approaches adopted in different countries is also presented. In addition, a brief overview of potential additional considerations and their influence on the selection of commonly applied techniques is provided, followed by the overall conclusions and recommendations.

Renovation and reinforcement techniques are grouped in three key families of failure mechanisms of concern: internal erosion, external erosion, and slope instability (Figure 2.9). To support practitioners in the selection of methods, these are first grouped by the family of mechanisms addressed, and subsequently by physical principle and functionality.

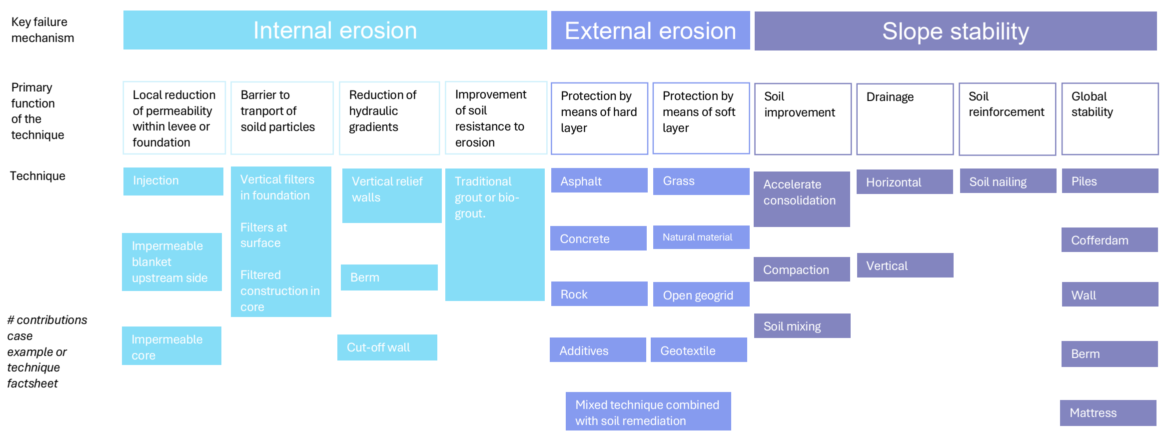

Figure 2.9 Overview of principles for techniques that can be used for retrofitting and reinforcement.

Damage caused by animal burrowing or vegetation is also a concern, and techniques specifically targeting these issues are therefore included. This overview aims to be as comprehensive as possible; however, specific site conditions or emerging approaches may lead to additional techniques that are not covered in the report.

2.4.1 *Internal erosion*

Table 2.1 addresses internal erosion mechanisms. Some techniques influence multiple mechanisms; where this is the case, this is indicated in the tables. The tables also provide references to sections in Part II, where case examples and technique factsheets – prepared by the members of the TC201 and their network – are presented. These sections further include references to design guidelines, manuals, and other relevant literature.

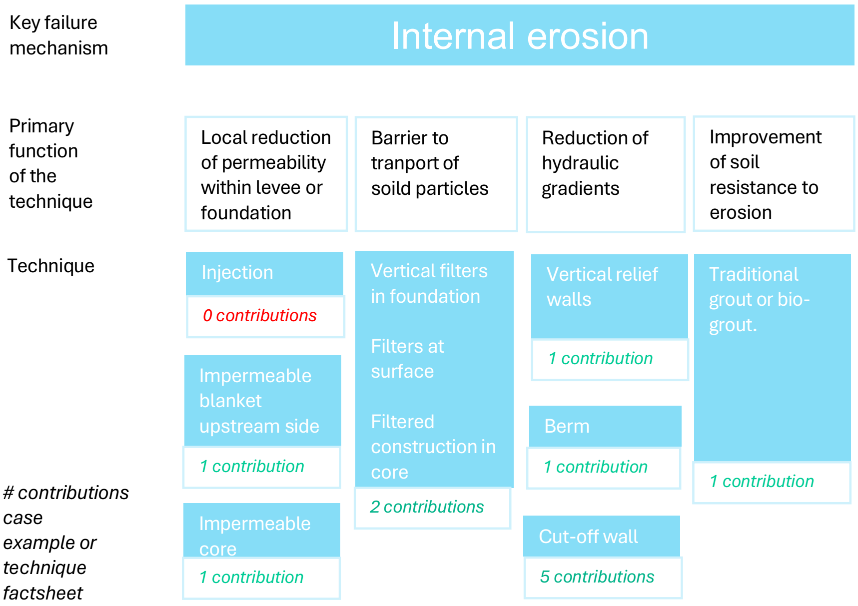

Figure 2.10 Overview of techniques available for failure mechanisms of internal erosion.

Table 2.1 Overview of techniques available for failure mechanisms associated with internal erosion. Techniques indicated with a star (*) are also effective for other mechanisms such as slope stability by reducing pore-water pressures.

Function	***Physical principle:***	***Remedial measure***	***Contribution (type case/technique factsheet, name & country corresponding TC member)***
Local reduction of permeability within the levee body	Reduce hydraulic gradients downstream of an internally treated zone to minimise soil-particle detachment and the initiation of internal erosion.	Injection techniques	-
	Reduce hydraulic gradients in the whole levee body to minimise soil-particle detachment and the initiation of internal erosion. *Also reduce hydraulic gradients at the downstream toe or landward side to minimise the initiation and progression of backward erosion piping.*	Impermeable blanket on the upstream side of the levee • Clay layer • Geosynthetic layer	- *Also mentioned in **case study (section 3.1.2)** on remediation measures against seepage in Japan.*
	Reduce hydraulic gradients downstream an impermeable core to minimise soil-particle detachment and the initiation of internal erosion. *Also reduce hydraulic gradients at the downstream toe or landward side to minimise the initiation and progression of backward erosion piping*	Impermeable core*	*Also mentioned in **case study (section 3.1.2)** on sheet pile cut-off wall, impermeable sheet and drainage at the Yabe River Levee*
Abating hydraulic gradients or providing barrier to particle transport	Prevent soil particles from being transported	Vertical filters in foundation Downstream surface filters Filtered construction in core*	**Case study *(section 3.1.5)* & Technique factsheet *(section 3.2.4)*** on vertical filtration & drainage in the foundation (along the Alpine Rhine). *By Gregor Portmann (Switzerland)* *Also mentioned in **case study (section 3.1.2 and 3.1.3)** on sheet pile cut-off wall, impermeable sheet and drainage at the Yabe River Levee*

Reduction of hydraulic gradients (by increasing seepage length or abating pore-water pressures)	Reduce hydraulic gradients in near-vertical flow at the landward side to minimise the initiation and progression of backward erosion piping	Vertical relief wells	**Case study *(section 3.1.1)* & Technique factsheet *(section 3.2.1)*** on relief wells (along the Mississippi River). *By Bryant Robbins (USA)*
	Decrease hydraulic gradients in the levee body and its foundation by increasing seepage length through the levee body or its foundation *Also reduce hydraulic gradients at the downstream toe or landward side to minimise the initiation and progression of backward erosion piping instability triggered by pore water pressure build-up at the toe.*	Berm*	- *Also mentioned in **case study (section 3.1.7)** on levee design in the Republic of Korea*
	Impervious components with the levee body to increase drainage length and reduce hydraulic gradients downstream including at the levee toe, to minimise the initiation and progression of backward erosion piping. Physical barrier to prevent soil particles from being transported.	Cut-off wall • Plastic • Steel • Soil mixing	**Case study *(section 3.1.3)*** on sheet pile cut-off wall, impermeable sheet and drainage at the Yabe River Levee. *By Hirotoshi Mori (Japan)* **Case study *(section 3.1.4)*** on sheet pile wall water side along the Danube River. *By Edina Koch (Hungary)* **Technique factsheet *(section 3.2.3)*** sheet pile walls. *By Edina Koch (Hungary) & Hirotoshi Mori (Japan) (combined contribution)* **Technique factsheet *(section 3.2.2)*** on deep soil mixing techniques for a cut-off wall. Based on *CFBR*. *Also mentioned in the **case studies** on remediation measures against seepage in Japan **(section 3.1.2)** and levee design in the Republic of Korea **(section 3.1.7)** .*

Improvement of soil resistance to internal erosion	Increasing soil cohesion, thereby preventing the detachment and loss of soil particles from the levee body or its foundation.	Traditional grout or bio-grout.	**Case study *(section 3.1.7)*** on levee design in the Republic of Korea & **Technique factsheet *(section 3.2.5)*** Xanthan gum biopolymer-based soil treatment. *By Ilhan Chang (Korea).*
Prevent formation of localised water flow pathways	Protection against burrowing	Mesh on land side slope	**Case study *(section 3.1.6)*** on treatment of the land side slope with mesh against burrowing animals. *Based on CFBR.*
		Sacrificial layer	-

2.4.2 External erosion

Table 2.2 addresses external erosion mechanisms. Some techniques influence multiple mechanisms; where this is the case, this is indicated in the tables. The tables also provide references to sections in Part II, where case examples and technique factsheets – prepared by the members of the TC201 and their network – are presented. These sections further include references to design guidelines, manuals, and other relevant literature.

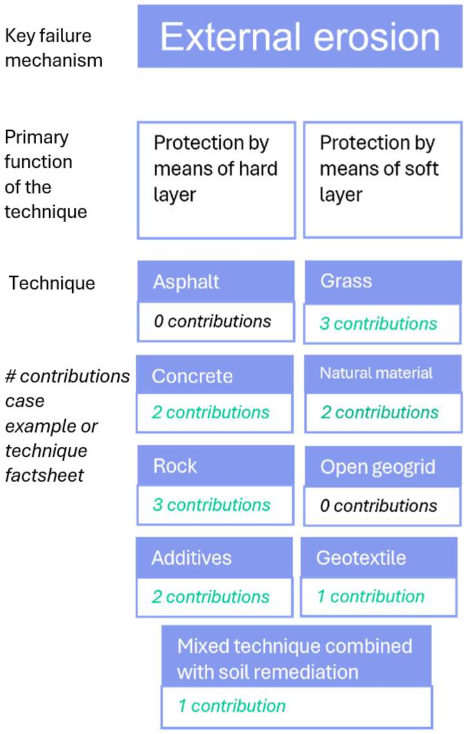

Figure 2.11 Overview of techniques available for failure mechanisms of external erosion.

Table 2.2 Overview of techniques available for failure mechanism external erosion.

Function	***Physical principle***	***Remedial measure***	***Contribution (type case/technique factsheet, name & country)***
Protection by means of hard layer	Absorb wave impact and protect against surface erosion.	Asphalt	-
		Concrete	**Case study (section 3.1.9) & Technique factsheet (section 3.2.6)** on geosynthetic honeycomb cells filled with concrete (for spillway at the Intabo levee). *By José Monteiro (Mozambique).* **Technique factsheet (section 0)** on shotcrete retaining walls. *Based on CFBR.*
		Rock • Riprap • Gabion wall • Paving stone	**Case study (section 3.1.10) & Technique factsheet (section 3.2.9)** on concreted rip rap (at the Rhône Delta). *By Thibaut Mallet (France).* **Technique factsheet (section 3.2.11)** on repair of embedded stone pitching. *Based on CFBR.* *Also mentioned in* ***case study*** **(section 3.1.9)** *on geosynthetic honeycomb cells filled with concrete for spillway at the Intabo levee.*
		Additives • Lime	*Also mentioned in* ***case studies*** *on concreted rip rap at the Rhône Delta* **(section 3.1.10)** *and mixed slope protection techniques as part as a soil remediation measure* **(section 3.1.18)**.
Protection by means of soft layer	Increase strength against surface erosion.	Grass	**Technique factsheet (section 0)** on external protection of embankment by weeding technique. *Based on CFBR.* *Also mentioned in* ***case studies*** *treatment of the downstream slope with mesh against burrowing animals* **(section 3.1.6)** *and mixed slope protection techniques as part as a soil remediation measure* **(section 3.1.18)** *.*
		Natural material	**Technique factsheet (section 3.2.8)** on control matting made from Swiss wood wool. *Also mentioned in* ***case study*** **(section 3.1.6)** *on treatment of the land-side slope with mesh against burrowing animals. Based on CFBR.*
		Open geogrid	-
		Geotextile	*Mentioned in* ***case study (section 3.1.7)*** *on levee design in the Republic of Korea.*

2.4.3 *Slope stability*

Table 2.3 addresses slope stability mechanisms. Some techniques influence multiple mechanisms; where this is the case, this is indicated in the tables. The tables also provide references to sections in Part II, where case examples and technique factsheets – prepared by the members of the TC201 and their network – are presented. These sections further include references to design guidelines, manuals, and other relevant literature.

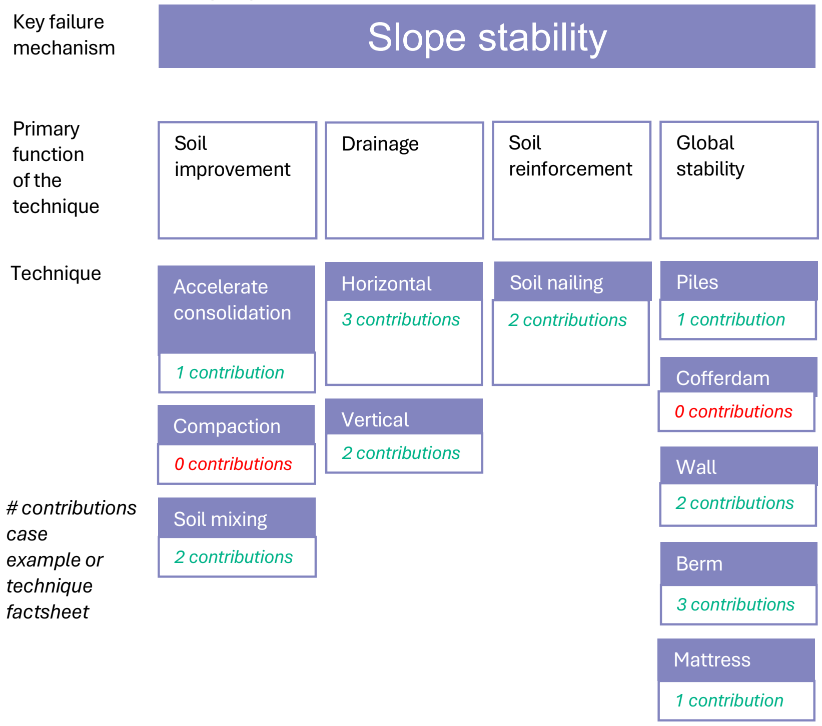

Figure 2.12 Overview of techniques available for failure mechanisms of slope stability.

Table 2.3 Overview of techniques available for failure mechanism slope stability. Techniques indicated with an asterisk (*) can also be effective for other mechanisms such as internal erosion.

Function	***Physical principle***	***Remedial measure***	***Contribution (type case/technique factsheet, name & country)***
Soil improvement	Improve soil shear strength.	Accelerate consolidation • Wick drains • Vacuum	**Case study (section 3.1.12) & Technique factsheet (section 3.2.13)** on increase of subsoil strength with vacuum consolidation (at the Markermeer levee). *By Jeroen Dijkstra (the Netherlands).*
	Improve soil shear strength.	Compaction	-
	Improve soil shear strength	Soil Mixing • Columns • Blocks	**Case study (section 3.1.20) & Technique factsheet (section 3.2.18)** on river levee stabilization with soil mixing (along the Scheldt River). *By Leen Vincke (Belgium)* *Also mentioned in **case study (section 3.1.7)** on levee design in the Republic of Korea. See also **technique factsheet (section 3.2.2)** on deep soil mixing techniques for a cut-off wall.*
Drainage	Reduce pore water pressures in levee or foundation	Vertical drains* • Relief wells • Drains in embankment body	**Case study (section 3.1.14)** & **Technique factsheet (section 3.2.15)** vertical drainage in embankment body *By Edina Koch (Hungary)* **Technique factsheet (section 3.2.19)** on trench and toe drain systems. *Based on CFBR.* *See also **case study** (section 3.1.1) **& technique factsheet** (section 3.2.1) on relief wells for internal erosion.*
	Reduce pore water pressures in levee or foundation	Horizontal drains	**Case study (section 3.1.16)** drainage (and piles) for road embankment on a slope. *By Jeff Wang (Hong Kong)* *See also **case studies** on (Drainage) berm and sheet pile walls along the Adige River **(section 3.1.18)** and remediation measures against seepage in Japan **(section 3.1.2)**.*
Soil reinforcement	Increase soil tensional strength	Dike nailing • Passive • Prestressed	**Case study (section 3.1.15) & Technique factsheet (section 3.2.16)** on retaining by means of soil nailing (along the Zuid-Willemsvaart canal). *By Leen Vincke (Belgium)* **Case study (section 3.1.19)** on anchoring and nailing system 'JLD Dike stabilizers' at the Ringdijk Watergraafsmeer Amsterdam (slope stability). *By Jos Teeuw (the Netherlands)*

Global stability	Modify the failure mechanism or the extent of the potential failure surface	Piles	**Case study (section 3.1.16)** anti slip piles (and drainage) for road embankment on a slope. *By Jeff Wang (Hong Kong)*
		Cofferdam	-
		Wall* • Anchored/ not anchored • Continuous/partial	**Case study (section 3.1.17) & Technique factsheet (section 3.2.17)** on anchored sheet pile walls (along the Waal River). *By Meindert Van (the Netherlands)* *Also mentioned in **case studies** on levee design in the Republic of Korea **(section 3.1.7)** and (drainage) berm and sheet pile walls along the Adige River **(section 3.1.18)**.*
	Stabilise levee by increasing weight of the toe and or flattening the side slopes	Berm	**Case study (section 3.1.11) & Technique factsheet (section 3.2.12)** on revetment and berm using granular material (along the Tisza River). *By Edina Koch (Hungary)* **Case study (section 3.1.18)** on (Drainage) berm and sheet pile walls along the Adige River. *By Alessandro Tarantino (Italy)* *Also mentioned in **case study (section 3.1.7)** on levee design in the Republic of Korea*
		Mattress Bamboo	**Case study (section 3.1.13) & Technique factsheet (section 3.2.14)** on Bamboo-mattress embankment reinforcement (at the Semarang-Demak integrated giant sea dyke with toll road). *By Hendra Jitno (Indonesia)*

2.5 Analysis

The framework presented in Section 2.4 provides an overview of reinforcement and renovation techniques classified according to the dominant failure mechanisms they address. Building on this framework, the contributions collected in Part II enable a reflection on the extent to which these techniques are established or innovative, as well as on the emergence of approaches that explicitly use natural materials. The following section therefore analyses the techniques described in Part II, distinguishing between established techniques embedded in national guidelines or recommendations, more innovative or emerging approaches, and techniques that rely on natural or bio-based materials.

2.5.1 Reflection on innovative and established techniques

The contributions collected for this report reveal a clear distinction between techniques that are widely established in levee engineering practice and those that can be regarded as innovative or emerging. In addition, techniques with natural materials are discussed as a separate category.

2.5.1.1 Established techniques

The first group of techniques described in Part II are commonly applied in levee engineering practice. These also appear in national guidelines, international documents such as the International Levee Handbook. Their design that falls under the broader categories of "embankments" and "earthworks" in Eurocode 7.

For internal erosion and seepage control, established techniques include relief wells, steel sheet pile cutoff walls, impermeable blankets on the riverside of the levee, and filtration and drainage systems. These measures are consistently applied in national practice in countries such as the United States, Japan, Hungary, Switzerland, and the Netherlands, and are reflected in national guidelines and recommendations for levee design and safety assessment. Their primary function is to reduce hydraulic gradients, via modifying the pore water pressure regime, increase seepage length, or provide controlled drainage paths, thereby mitigating the risk of backward erosion piping and related failure mechanisms.

For external erosion, commonly applied techniques include riprap (either loose or concreted), concrete or shotcrete revetments, and conventional grass covers. These solutions are intended to protect levee slopes against hydrodynamic loading associated with wave action and overflow, and mitigate surface erosion.

For slope stability, commonly implemented techniques include stability berms, toe and trench drains, vertical drainage systems within the embankment body, and sheet pile walls.

2.5.1.2 Innovative or emerging techniques

In addition to the well-established techniques described above, Part II also includes a number of innovative or emerging solutions. These are techniques that have already been implemented in practice but are not yet widely adopted or fully embedded in guidelines and recommendations. Their development is often driven by constraints such as limited space, high material or construction costs, environmental objectives, or the need for adaptable solutions under increasing uncertainty related to climate change.

Examples of innovative techniques include biopolymer-based soil treatments for reducing permeability and improving soil strength, vacuum drains to accelerate consolidation and the stability of levee foundations, soil mixing shear walls oriented transversely to the levee axis, composite plastic cut-off walls, and actively prestressed anchoring and nailing systems.

Within this group of innovative techniques, the subgroup involving mixed-in-place methods or other soil treatment approaches deserve particular attention, as these techniques can address multiple failure mechanisms. Mixed-in-place structures, used as a longitudinal elements, are installed as a cut-off walls to mitigate backward erosion piping. Soil mixing walls are also applied to address slope stability failure mechanisms; in this case, the soil mixing shear zones are oriented transversely to the levee axis.

Soil treatment involving the addition of biopolymers binding agents can be used to mitigate internal erosion but also to encapsulate pollutants, as discussed in Part II of this report. Techniques based on natural or bio-based materials are worth mentioning. Their primary added value lies in reduced environmental impact, improved ecological integration, and the use of locally available materials. Widely adopted materials include stones and vegetation-based erosion protection. In Part II of this report, techniques with bamboo mattresses, coconut and jute matting, and wood wool erosion protection are also included. However, their applicability is generally context dependent, and they are most often used as complementary rather than standalone reinforcement measures.

2.5.2 *Comparison of design approaches for specific techniques across countries*

Some of the case studies and factsheets presented in Part II describe similar techniques, but differ in their design approach, application or material use. A comparison of these techniques, including cut-off walls, filtration and controlled drainage, and soil strengthening through additives, is presented below.

2.5.2.1 (Cut-off) walls as a technique for internal erosion and slope stability

Cut-off walls are a widely applied technique to mitigate internal erosion and piping. Differences across countries include the location of the cut-off wall within the levee cross-section, the materials used, the construction methods applied, and whether the cut-off walls are continuous or suspended, i.e., whether they reach or do not reach an impermeable layer, respectively.

There are some variations regarding the location of the cut-off wall. In the case studies from Japan and Hungary, cut-off walls are located at the riverside toe of the levee. In contrast, the French, Italian, and Korean factsheets show cut-off walls installed from the levee crest, extending downward through the foundation. A third configuration consists of cut-off walls installed at the landside toe or berm. The choice of the location of the cut-off wall is influenced by the dominant failure mechanism (internal erosion manifesting as suffusion/suffusion/backward erosion piping and/or slope stability of the embankment body), reduced interference with river training works and/or urban settings, available space, accessibility, and other construction constraints.

The factsheets in Part II of this report indicate the use of a range of materials for the cut-off walls, reflecting both historical practice and more recent developments. Steel remains the most widely used material for the sheet piles and is the material used in the Japanese and Hungarian case studies. In addition to steel, concrete sheet piles, plastic sheet piles and mixed-in-place (soil mixing) walls are described in several factsheets from France, Japan, Italy, and Korea. The Korean contribution mentions a hybrid cut-off wall made of vinyl sheet piling with gasket and cement-soil mix barrier and composite PP-based plastic cut-off walls. The mixed-in-place walls are mentioned in the French, Italian, and Japanese case studies.

Depending on the material of the cut-off wall, variations in construction methods may also arise. Steel and plastic sheet piles are generally installed using vibratory driving or pressing techniques, depending on soil resistance and vibration constraints. For mixed-in-place cutoff walls, construction relies on soil mixing techniques, such as cutter soil mixing or deep soil mixing. The Japanese contribution also mentions slurry trench with an option to include a soft vinyl chloride sheet as a watertight material to improve watertightness.

A further distinction is made between continuous cutoff walls and suspended cutoff walls to mitigate backward erosion piping. Continuous cutoff walls extend down to a low permeability layer and aim to fully block under-seepage through the foundation layers. In contrast, suspended cutoff walls are applied where no clearly defined impermeable layer is present at reasonable depth. In such cases, the cutoff wall only partially penetrates the foundation layers, increasing the seepage path length and reducing hydraulic gradients without fully cutting off flow. Sometimes, suspended cutoff walls are therefore combined with other measures, such as berms or drainage systems, to achieve the required level of safety.

For walls mitigating the problem of slope instability, these could also be combined with anchoring to achieve the required level of safety, allowing the use of thinner steel profiles and/or extending the design service life of the structure. The Dutch contributions mention the application of anchored (steel) sheet pile walls in a case where space was limited.

2.5.2.2 Filtration and controlled drainage as seepage control to target pore water pressure for internal erosion and slope stability

In addition to cut-off walls, several contributions in Part II describe measures that rely on filtration and controlled drainage mitigate internal erosion and/or slope stability. These measures include relief wells, vertical filters, and horizontal and vertical drainage systems. Although they share a common functional principle, cases across countries illustrate different design emphasis and combinations of measures.

Relief wells are presented in Part II primarily through the extensive case study description from the United States. The Mississippi River case illustrates an approach in which relief wells are designed as a primary seepage control system, intercepting under-seepage through a thick pervious aquifer overlain by a blanket layer. For relief wells, this upper confining blanket should be no thinner than about 5 feet (1.5 meters). The design focused on well spacing, filter configuration, and long-term maintainability. Relief wells are also shortly mentioned as a considered design alternative in Hungary for internal erosion, and as a variation of the toe drainage drench in France, although uncommon. In the case study from Liechtenstein, vertical gravel columns were installed to relieve pore-water pressures during a flood event.

Filtration measures are also installed in the embankment body and in the berm to prevent soil particles from being transported. To reduce the pore water pressures in the levee body, vertical drainage systems are used, as described in one of the Hungarian cases, where vertical gravel drains are installed within the levee to accelerate dissipation of excess pore pressures that develop during prolonged floods. Also, drainage can come from constructing a filtered berm from coarse materials such as gravel. Drainage can also be achieved by constructing a filtered berm composed of coarse-grained materials such as gravel. The coarse-grained material and an optional drainage system provide seepage control along the landside levee toe, which are recurring solutions in one of the Hungarian cases and cases from Italy and Liechtenstein.

2.5.2.3 Use of additives for internal erosion, external erosion, and slope stability

Part II includes a limited number of case studies and factsheets that address the use of additives, to reduce erodibility or enhance shear strength.

One emerging approach described in Part II concerns the use of bio-grout or biopolymer-based additives to increase soil shear strength and resistance to internal erosion. The technique factsheet on xanthan gum biopolymer-based soil treatment, together with the associated Korean case study, illustrates how additives can be mixed into the soil to reduce permeability and also limit particle detachment under seepage flow.

Lime is also referenced as an additive to improve the erosion resistance of surface layers, notably in the case studies on concreted riprap at the Rhône Delta and on mixed slope protection techniques as part of soil remediation measures. In these cases, the additive does not function as a stand-alone reinforcement measure but rather as an enhancement of conventional protection layers, improving resistance to surface erosion during overflow or wave attack.

A further category of soil reinforcement though additives is represented by soil mixing techniques, where binding agents such as cement or lime are mixed in situ with the native soil. Although soil mixing is discussed extensively in Part II in the context of cut-off walls and shear walls, it also represents a form of soil strengthening by additives when the primary objective is to improve mechanical performance rather than to create an impermeable barrier.

Such treatments are particularly attractive where traditional measures are difficult to implement due to space constraints or where targeted treatment of vulnerable zones is desired. At the same time, the limited number of documented applications highlights that long-term performance, durability, and upscaling remain important considerations.

2.5.3 Influence of local characteristics on the selection of techniques

For a specific failure mechanism, the selection of a type of technique will be influenced by local characteristics of the subsurface and the environment. For instance, for the internal erosion failure mechanism of backward erosion piping, the available space to increase the seepage length, the thickness of the blanket in the protected side, and the thickness of the aquifer are important features determining which techniques are effective.

Where sufficient space is available on the landside, increasing the seepage length by means of a berm is a robust and well-established option. If possible, techniques to reduce the hydraulic load, such as elongation of the seepage path in the foreshore or injection techniques to reduce the permeability in the aquifer (e.g. SoSeal) should be considered. Where space is limited by adjacent infrastructure or urban development, the most effective measures are dependent on the thickness of the aquifer and the cover layer. In situations where a pervious aquifer is overlain by a relatively thin blanket, and controlled discharge of seepage water is acceptable, relief wells, drainage systems, or filter measures such as vertical sand-tight geotextiles, coarse sand barriers and plastic filter screens are effective alternatives. For thick aquifers, suspended cut-off walls, made of steel, plastic, soil-mix, mix-in-place or trisoplast remain widely used options in the Netherlands.

2.6 Additional considerations for selection of techniques

For a specific levee that needs reinforcement or renovation, local characteristics and other design considerations or long-term uncertainties may play a role.

Table 2.4 provides an overview of key considerations and describes how these may influence the performance of a cut-off wall, a sheet pile wall used for stability, a berm designed to mitigate slope sliding, and drainage measures.

Table 2.4 Set of considerations and how they influence the selection of levee mitigation techniques.

Consideration aspect	***Berm***	***Cut off wall***	***Sheet pile wall for stability***	***Drainage***
Adaptation of the levee to increasing flood frequency, intensity, or duration due to climate change.	*Could be extended if space allows*	*If the wall is continuous, the measure remains affective with increased hydraulic loading.* *If the wall is suspended, it cannot be easily deepened*	*It cannot be easily deepened*	*Capacity could be increased*
Sensitivity to land subsidence	*Berm will also subside and possibly induce additional subsidence on soft soil*	*Cutt off walls modify the pore-water pressure regime in the foundation layers and potentially generate additional settlements or heave on the riverside and landside of the wall*	*If wall is founded in granular layer, the subsiding soil might generate negative friction on the wall*	*Drainage of soft soil might exacerbate subsidence*
On-site constructability / Ease of construction	*Requires large volumes of soil and significant transport and expropriation costs*	*Requires specialized mixing rigs or trenching equipment. Complexity depends on depth of installation and soil conditions.*	*Requires piling machinery and access for long steel elements. Noise and vibration may impose constraints in urban areas.*	*Generally easier to install with light equipment. Flexible placement but requires precision in filter installation*
Considerations of emissions (CO2, …)	*Emissions primarily as a consequence of transport. Use of locally-sourced soil can contribute to reduce emissions*	*Different materials can be considered such as mixed in place low-carbon concrete, plastic,*	*Relatively high CO_2 emissions for steel,*	*Relatively low emissions. For active drainage, emissions associated with power required for pumping to be considered*
Consideration of changing material prices	*If material is not locally available, sensitive to material prices*	*Different materials can be considered.*	*Sensitive to high steel prices*	*Low sensitivity to material prices.*
Multi-functional levees	*Berm can be designed for multiple functions*			

Table 2.5 Set of considerations and how they influence the selection of levee mitigation techniques (continuation).

Consideration aspect	***Berm***	***Cut off wall***	***Sheet pile wall for stability***	***Drainage***
Possibility to monitor the state/effectiveness of the measure	*Deformations of berm can be observed*	*Monitoring pore water pressures on the two sides of the wall is possible.*		*Monitoring of effect on pore water pressure regime is possible.*
Environmental or biodiversity considerations	*Affects larger area. However, could be used to improve biodiversity e.g. with biodiverse grass cover*	*It requires little space. Concerns may arise regarding the introduction of non-natural materials into the environment (e.g. potential release of micro- or nanoplastics). impacts on groundwater quality and flow may affect flora and fauna, particularly in protected or environmentally sensitive areas*	*Requires little space*	*Influence on groundwater might affect flora and fauna in protected area*
End of life, possibility to remove	*Relatively easy to excavate*	*Difficult to remove, in particular plastic or soil mix*	*Often removed as temporary supporting structure, Difficult to remove for long term applications or in combination with other structures*	*Relatively easy to remove*

2.7 Conclusions and recommendations

This report has provided an international overview of established and innovative techniques currently applied for the reinforcement and renovation of levees across different countries, with a particular focus on geotechnical failure mechanisms. Respondents to a survey conducted within the TC201 member network indicated that primary failure mechanisms of concern are internal erosion, external erosion, and slope instability. They also documented remedial measures currently in practice through the presentation of case studies and the provision of factsheets for individual techniques.

Many respondents indicated that existing national and international guidelines and recommendations (e.g., the International Levee Handbook [CIRIA 2013]) only partially meet their needs, particularly with regard to innovative methods for which limited guidance is available. Respondents also indicated that knowledge of methods to mitigate the negative effects of animal burrowing or vegetation on levee performance is limited and remains a concern.

The still-emerging expertise on innovative methods is reflected in the smaller proportion of case studies and factsheets dedicated to innovative or emerging techniques. Examples included in this report encompass mix-in-place walls and vacuum consolidation for improving stability against slope sliding, as well as approaches aimed at modifying soil properties or improving resistance, such as soil treatment with additives or bio-based materials.

A first recommendation arising from this report is that the long-term performance of these innovative techniques should be closely monitored in order to further assess their robustness and applicability.

A second recommendation arising from this report is for the professional and academic levee community to invest further in the development, testing, and application of innovative methods for levee renovation and reinforcement.

There are several drivers for the development and application of innovative techniques, including increasing pressure on available space, rising material and construction costs, and growing consideration of biodiversity and carbon emissions.

In the longer term, the impacts of climate change, resulting in more variable and less predictable loading conditions on levees, are expected to increase demand for techniques that can be more easily extended, modified, or adapted to address these uncertainties.

Given the increasing demand for space, multifunctional levees that also deliver social or ecological benefits are attracting growing interest. Examples include levees that provide recreational space or enhance biodiversity through the use of more diverse grass and vegetation covers. This calls for innovative methods that combine robust structural performance of levees under hydraulic loading with environmental, social, and biodiversity benefits.

Part II: Case descriptions and technique factsheets

Contributions by members of the
Technical Committee on Geotechnical Aspects of Dikes and Levees
(TC201)

Editors Part II:
Core team of the TC201

3 Part II: Case descriptions and technique factsheets

In this part, chapters will be included showing the case descriptions and the technique factsheets. If there are technique factsheets in combination with a technique, a reference of the technique factsheet be made in the chapter describing the case and vice versa.

If multiple contributions are available for a technique factsheet, these will be combined, showing the specifics of both sheets.

Case descriptions

Contributions by members of the
Technical Committee on Geotechnical Aspects of Dikes and Levees
(TC201)

3.1 1 Case description: Relief wells along the Mississippi River (internal erosion)

Authors: Nicholas Bidlack, USACE, John “Ben” Tatum, USACE, Nicholas “Nikko” Aleman, USACE
Keywords: Backward erosion piping (BEP); river levees; impervious top stratum; pervious substratum; relief wells

For the corresponding technique factsheet on relief wells, please refer to Section 3.2.1.

Setting

Levees along the Mississippi River from Cape Girardeau, Missouri down to the Gulf are part of the Mississippi River and Tributaries (MR&T) project. In the wake of the devastating 1927 flood, it was deemed necessary to put into place a comprehensive, unified system of infrastructure within the lower Mississippi Valley that would provide unprecedented flood risk management. There are approximately 1,600 miles of levee along the Mississippi River in the lower Mississippi Valley. Since 1940, the lower Mississippi River levees have been studied for backward erosion piping (BEP) through the foundation. The initial efforts were prompted by the occurrence of heavy seepage and sand boils along numerous reaches of these levees during the 1937 flood.

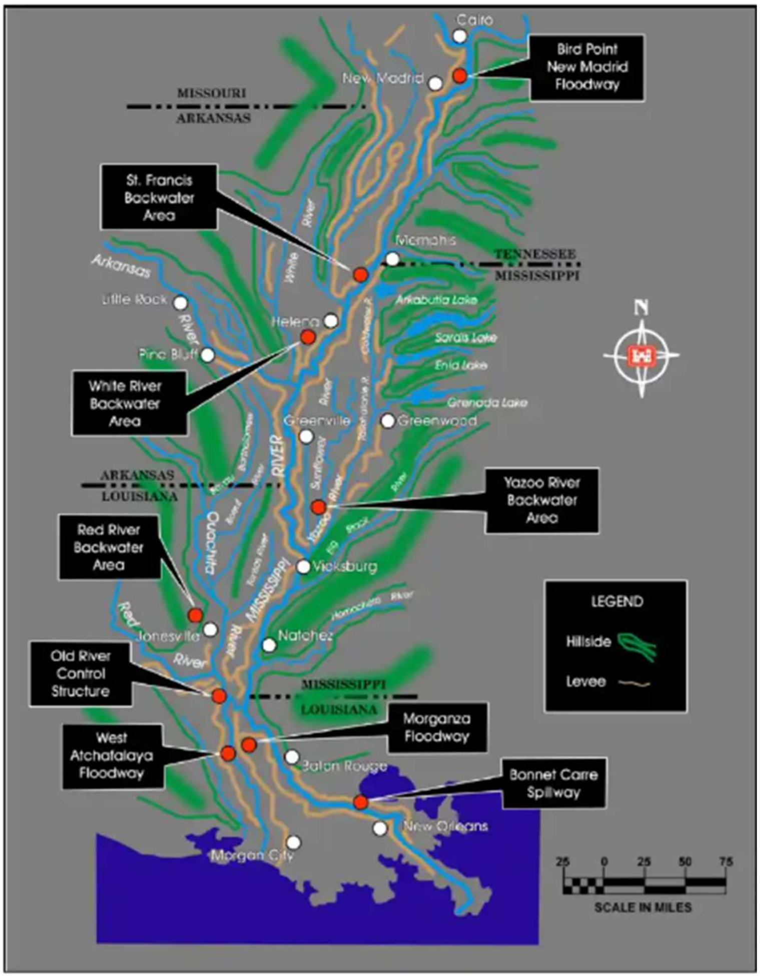

Figure 3.1 General location of levees within the MR&T Project

The lower Mississippi River levee subsoils typically consist of a relatively impervious top stratum (clays and silts) when compared to the pervious substratum (sands and gravels). The fine-grained, upper stratum is typically referred to as the “blanket”. The pervious substratum ranges from 100 feet thick near Memphis, Tennessee to 180 feet thick near Sikeston, Missouri with a Tertiary clay deposit as the lower confining layer.

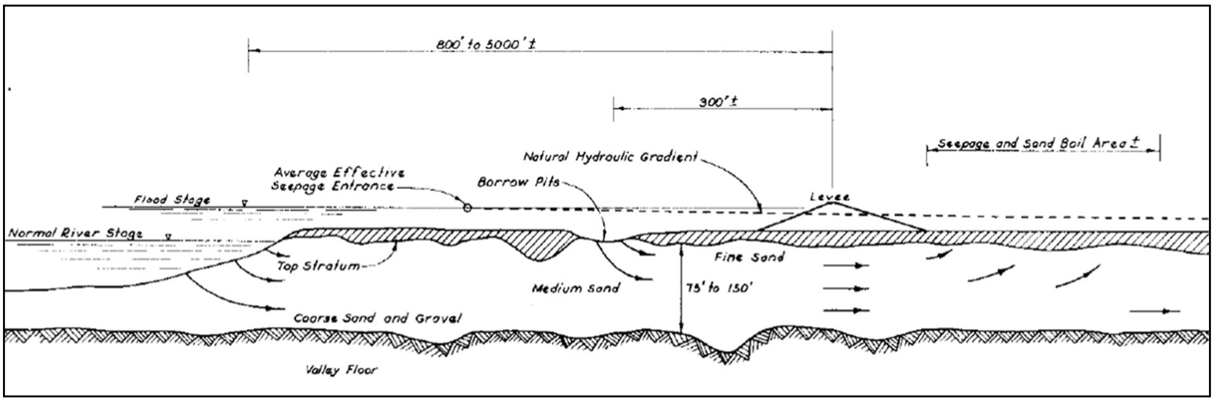

Figure 3.2 - Generalized Mississippi River Levee Cross-section and Foundation Condition from TM 3-424 (1956)

Problem description

Seepage and sand boils landward of Mississippi River levees have been observed during major high-water events dating back to the 1937 flood and are key indications of backward erosion piping (BEP) through the levee foundation. In general, backward erosion piping of the foundation initiates with foundation underseepage leading to heave, which occurs when the upward forces acting on the bottom of the blanket exceed the downward forces of the blanket. Erosion of the foundation materials can progress when horizontal gradient exceeds a critical horizontal gradient for the foundation materials. This critical horizontal gradient is typically a function of material classification and particle size distribution. As the erosion continues, the low strength zone underneath the levee can collapse resulting in breach and uncontrolled release of water behind the levee.

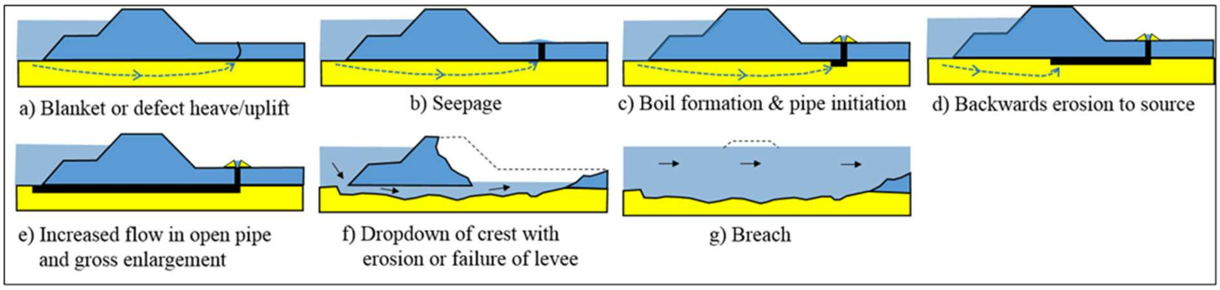

Figure 3.3 - Backward Erosion Piping Through the Foundation (Adapted from van Beek et al. (2010))

Remediation measure(s)

Description the selected remediation methods

Relief wells have been installed at the landside toe of the lower Mississippi River to reduce uplift pressure and similarly reduce the likelihood of sand boils and piping of the foundation materials. Relief wells accomplish this by intercepting underseepage groundwater on the landside of a levee and providing a properly filtered exit for underseepage without movement or erosion of the foundation material.

Relief wells have been successful where landside upper confining layers (impervious top stratum) are present with underlying pervious substratum as typically found along the lower Mississippi River. Without an upper confining layer at the levee toe at least 2-3 orders of magnitude lower in permeability than the aquifer, there may not be enough artesian pressure to push enough water out of the relief well to yield a significant reduction in head. The levees in the lower Mississippi River Valley have maximum flood loadings typically ranging from 15 to 35 feet. Relief wells are typically not considered an economically advantageous remediation method when well spacings get less than 50 to 75 feet (center-to-center spacing). This typically requires that upper confining blanket is no thinner than about 5 feet. Excessive seepage gradients in areas without a confining layer are best remediated using other means (e.g. cutoffs, berms, etc).

Figure 3.4 - Relief Wells flowing during high water 2016 in Arkansas

The first applications of relief wells were installed along the Mississippi River near Commerce and Trotters, Mississippi and at Wilson Point, Louisiana in December of 1942 and January of 1943. Currently there are over 4,200 relief wells in place along lower Mississippi River levees.

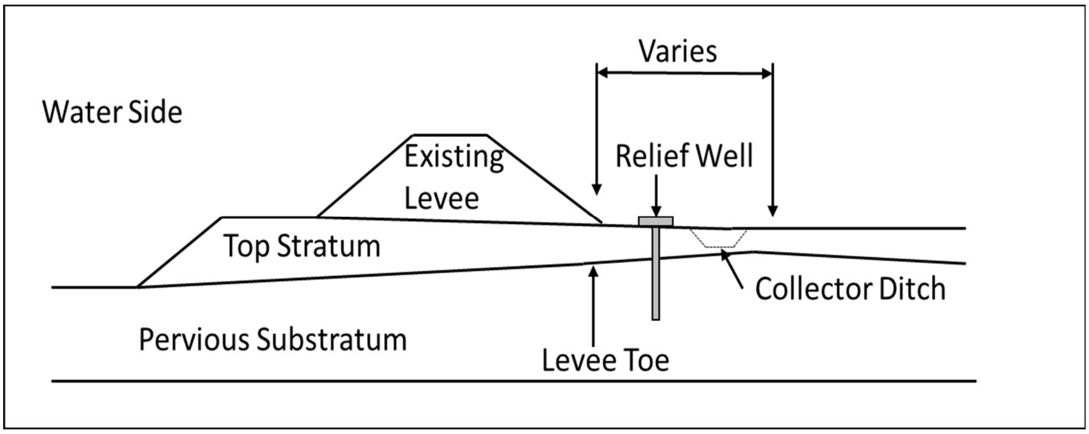

Figure 3.5 - Typical Cross-section with Relief Wells

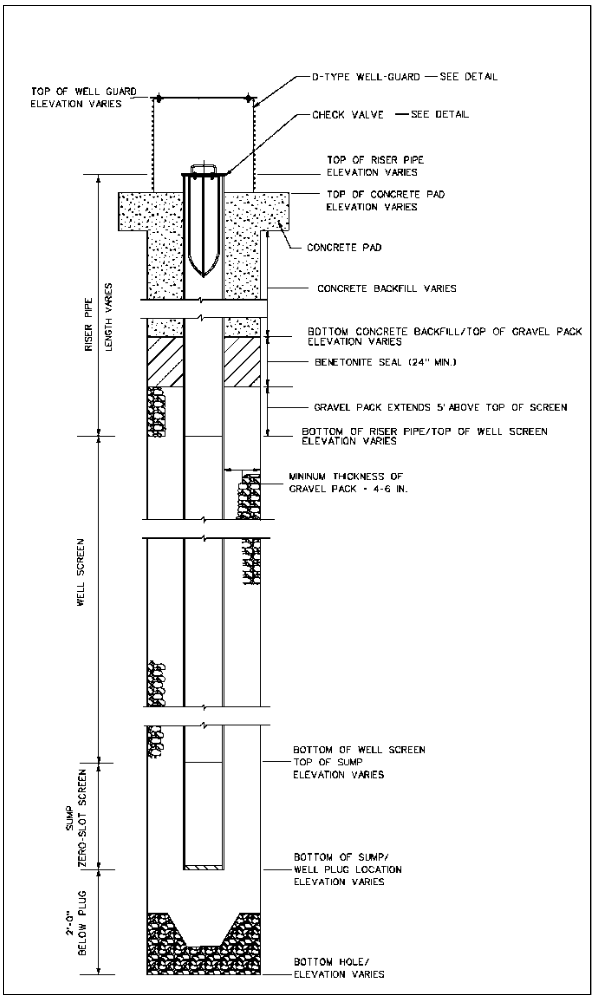

Figure 3.6 - Typical Relief Well Design Detail

Performance

Relief wells were developed and extensively implemented within the MR&T project as a critical countermeasure to underseepage problems like formation of sand boils and piping of foundation materials, with numerous examples of successful performance during high-water events supporting their efficacy when properly designed, installed, and maintained. Currently there are over 4,200 relief wells along levees within the MR&T project. Many of which were in place during the 2011 flood event and integral in the system of levees passing the historic event.

Other information about this case

Maintaining relief wells is essential to ensure their continued proper functioning and the successful operation during high water events. Proper operation and long-term maintenance are crucial considerations, as the robustness, redundancy, resiliency, and overall effectiveness of relief well systems depend on routine and proper upkeep.

Routine activities are necessary to keep wells functioning properly and prevent degradation. This involves prompt removal of any trash or obstructions that may enter the well or well guard. Accumulated sand or other material around well discharge areas should also be removed. Check valves, which prevent backflow of surface water and contamination, and flap gates must be regularly checked, cleaned, and maintained to operate properly and close securely. Biofouling (biological encrustation) and mineralization of the screen, filter pack, and aquifer surrounding the well filter can also reduce efficiency of the wells.

Pump testing is a direct measure of relief well performance and is generally recommended for all wells every 5 years. Pump tests are used to determine Specific Capacity Ratio, efficiency, or changes in entrance head loss.

Evaluation for maintenance and/or rehabilitation uses data from pump tests, video inspections, and field observations. Historically, a decline to 80% of the baseline SCR or Efficiency often triggered maintenance. Decreased well flow or increased piezometric levels during flood loading generally indicate clogging and require action if projected levels exceed design limits.

3.1 2 Case description: Remediation methods for seepage in Japan (internal erosion/slope stability)

Authors: Hirotoshi Mori, Shunsuke Sako, Japan
Keywords: Slope sliding, Piping; River levees; increasing hydraulic loads (intensity or frequency) due to climate change

Setting

River systems in Japan
River systems are classified into four categories according to the River Act in Japan as shown in Table 3.1. The four categories are described below:

a. 1st Class River Systems:
   River systems which are important for land management or national economy are managed by the central government, the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) and local government. There are 109 1st Class river systems consisting of about 14,000 rivers for about 12,000 km as shown in Figure 3.7.
b. 2nd Class River Systems:
   Some other river systems are managed by local governments. There are 2,711 2nd Class river systems consisting of about 7,000 rivers for about 36,000 km.
c. Provisional Class River Systems:
   Small-scale independent river systems or small rivers in 1st Class or 2nd Class River Systems are managed by local government or municipality. There are 2,577 Provisional Class River Systems consisting of about 14,000 rivers for about 20,000 km.
d. Non-classified River Systems:
   All the other small rivers are managed by municipality.

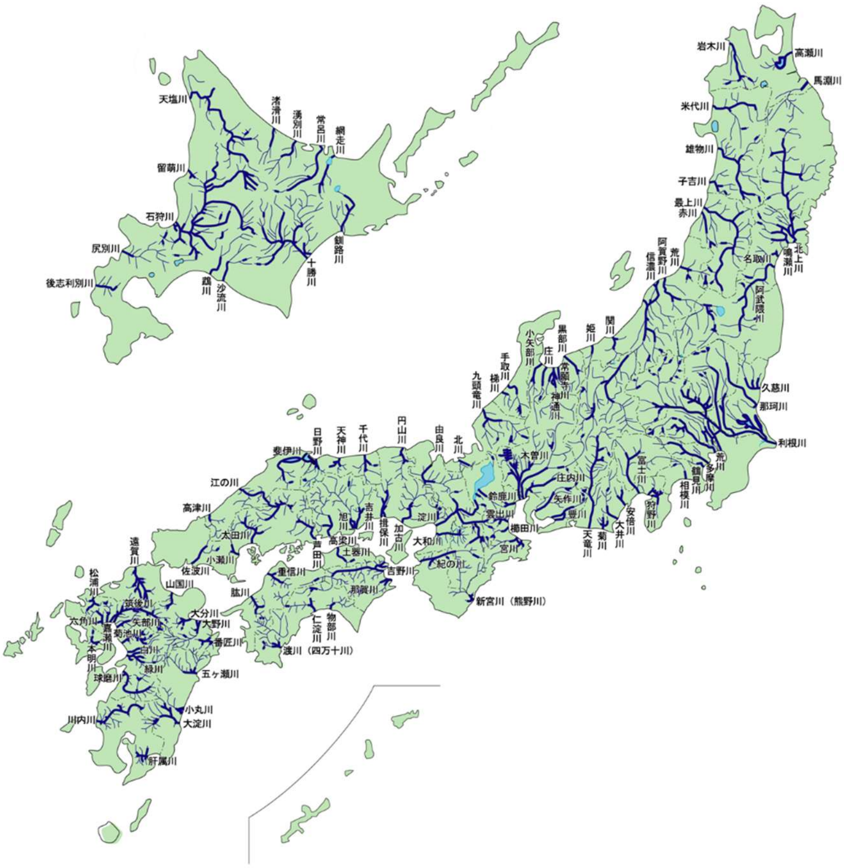

Figure 3.7 First class river systems (MLIT)

Table 3.1 Rivers in Japan

Category	Number of river systems	Number of rivers	River length (km)	Floodplain areas (km^2)
1st Class	109	14,062	11,758	240,741
2nd Class	2,711	7,080	35,867.2	107,041
Provisional class	2,577	14,320	20,102.9	-

The total length of 1st and 2nd Class River System, Provisional Class River System is 68,000 km.

The length of river levees in 1st Class River Systems managed by MLIT is 12541.4 km. 8,865.1 km of levees are completed as planned. For 3676.3 km of levee, the current width and height are less than planned. In addition, 897.3km of levees is planned for construction.

Historically, river levees were constructed with materials that could be obtained nearby, such as dredged silt or sand from rivers nearby. As a result, river levees have a broad range of soil types inside, including fine soil (clay or silt) and coarse soil (sand and gravel) constructed with various compaction method.

In Japan, floodplain areas cover 38,000 km2, about 10 % of the national land 377,700 km^2 and 1/3 of the total flat land areas. The floodplain areas have a population of 59 million people, almost 1/2 of the total population and have assets and infrastructures worth 550 trillion yen, about 75 % of the total assets and infrastructures.

Problem description

Seepage assessment in Japan

The assessment for seepage has been conducted to all the levees managed by MLIT as follows:

1. Design of river levees
   There is "the Technical Standard for River and Sabo: Design" by MLIT for the design and the safety assessment of river levees. The technical standard shows the standard design of not only river levees, but also lakeshore levees, seawalls, open levees and special levees which are made by concrete or metal.

   In addition to the technical standard, "the Design Guideline for River Levees" by JICE (the Japan Institute of Country-ology and Engineering) is published. The design guideline describes survey, design, safety assessment and the countermeasures against risks and events related to seepage, erosion and earthquake.

2. Safety assessment for a seepage failure
   The assessment by the design guideline follows the process below:

2.1. Select a representative cross section
   The assessment conducted based on "the representative cross sections" which are set as follows.
   The design guideline divides the levee into "the series of sections" based on the river's characteristics (e.g., design flood level, flood duration, etc.), topographical features, the conditions of protected areas. The sections are generally divided at points where a branch river diverges or merges or locations where levees are connected to a mountain.

   Next, a series of sections that can be considered to have similar soil layers and levee shape is subdivided into "the subdivided sections". The assessment uses the sections as the smallest unit.

   Finally, the representative cross section is set at each subdivided section. The cross section is selected at locations having the highest risk of seepage in the subdivided section.

2.2. Set a rainfall and river water level
   Rainfall and river water level need to be set for a seepage analysis shown in Figure 3.8 and Table 3.2.

   Total rainfall for the initial condition before flood is fixed as the mean monthly precipitation during the rainy season in an average year. The intensity is about 1mm/hr, while the summation is equal to the mean monthly precipitation.

   Total rainfall during flood is set as the planned rainfall of the river used for the river management planning. The intensity should be about 10 mm/hr while the summation is equal to the planned rainfall.

River water level varies depending on the characteristics of its basin and differs between the upstream or downstream of a river. Thus, the shape of river water level is calculated from a few river points based on the planned flood or the real floods. The maximum river water level is adjusted to the design high water level.

Table 3.2 Rainfall and river water level for the assessment (the Design Guideline for River Levees, 2012)

	Rainfall		River Water level
	Total rainfall	Intensity	
Initial	About the mean monthly precipitation during the rainy season in an average year	About 1 mm/hr	About the normal water level.
Flood	Planned rainfall of the river	About 10 mm/hr	Calculated from a few river points based on the planned flood or real floods. The maximum river water level is adjusted to the design high water level.

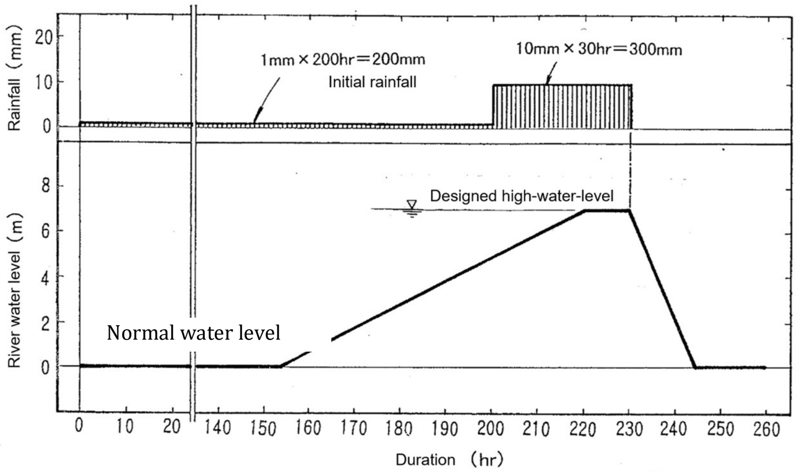

Figure 3.8 Example of rainfall and river water level (the Design Guideline for River Levees, 2012)

3. Geotechnical Modelling
   The model of levees for seepage and circular slip analysis represents the shape and the internal soil layers. The internal soil layers are modelled based on soil characteristics collected from a boring survey at three points: the levee crown and front and back slopes in Figure 3.9. A sounding survey is also conducted additionally.

   The soil parameters such as hydraulic conductivity, unsaturated hydraulic property, wet density, cohesion and internal friction angle, are set for each soil layer by soil tests or the design guideline.

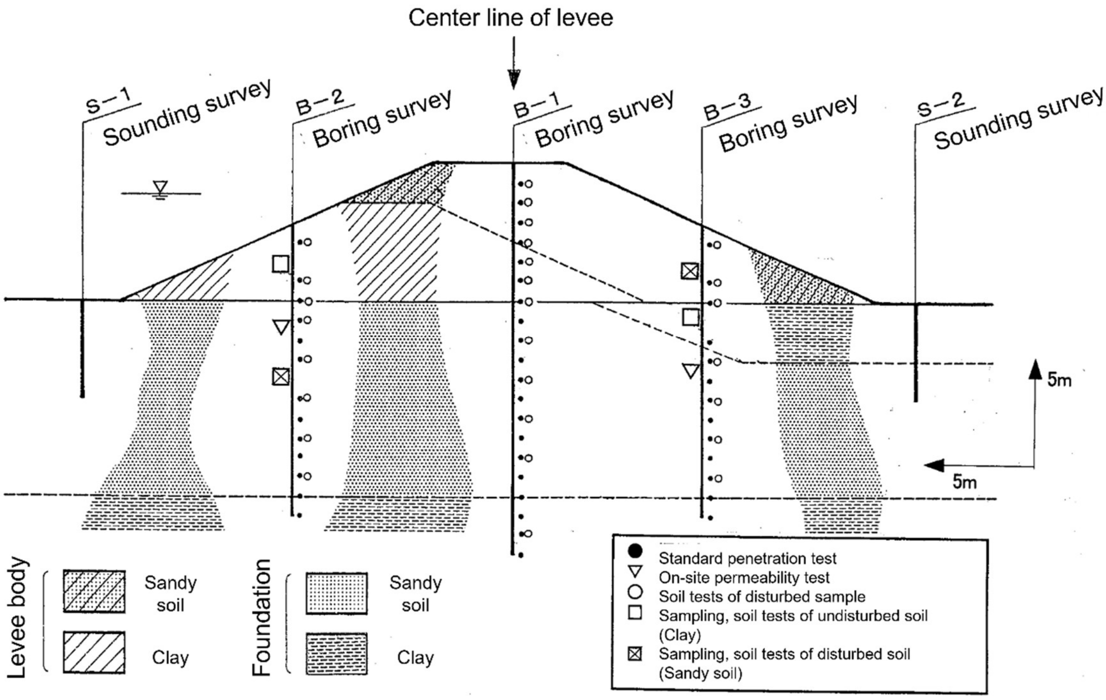

Figure 3.9 Example of a survey for soil layers (the Design Guideline for River Levees, 2012)

4. Seepage analysis and circular slip analysis
   Seepage analysis is used to assess the safety for piping and circular slip analysis is for slip failure.

   The unsteady saturated/unsaturated seepage analysis is conducted for the model with the rainfall and the river water level. The equation below shows the basic function for seepage analysis.

$$\frac{\partial}{\partial x}\left(k\frac{\partial \phi}{\partial x}\right)+\frac{\partial}{\partial z}\left(k\frac{\partial \phi}{\partial z}+k\right)=(C+\alpha\cdot S_s)\frac{\partial \phi}{\partial t}$$

   where,

   x: Horizontal axis of levee cross section (m)
   z: Vertical axis of levee cross section (m)
   k: Hydraulic conductivity (m/s)
   φ: Pressure head (m)
   C: Specific moisture capacity (1/m)
   α: Saturated zone in the case of 1, unsaturated zone in the case of 0
   Ss: Specific storage (1/m)
   t: Time (s)

   Figure 3.10 shows an example of seepage analysis with the result of experiment using a levee on site. The phreatic lines have developed to reach the back toe of a levee by time. The assessment for piping is conducted by the largest of local hydraulic gradients of the foundation ground near the toe or uplift force acting on the bottom surface of the covered soil.

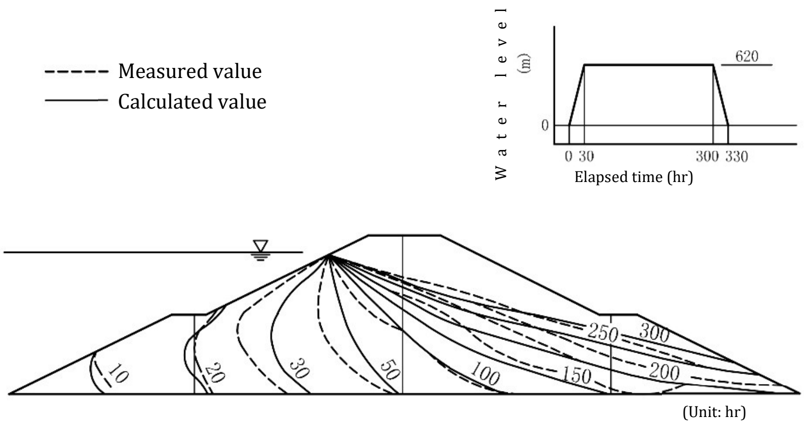

Figure 3.10 Example of a seepage analysis with the result of experiment (the Design Guideline for River Levees, 2012)

The circular slip analysis, the modified Fellenius method, is carried out by the equation below with using the phreatic line from the seepage analysis.

$$F_s = \frac{\sum\{cl + (W - ub) \cdot \cos\alpha \cdot \tan\phi\}}{W \cdot \sin\alpha}$$

Where,

Fs: Factor of safety
u: Pore water pressure of slip surface (kPa)
W: Weight of partitioned segment (kN/m)
c: Cohesion of soil along slip surface (kPa)
l: Length of arc (m)
φ: Internal friction angle of soil along slip surface (degree)
b: Width of partitioned segment (m)

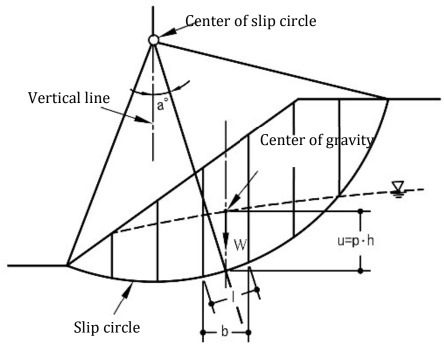

Figure 3.11 Circular slip analysis (the Design Guideline for River Levees, 2012)

The results of circular slip analysis are compared with the standard shown in Table 3.3.

Table 3.3 The standard of the assessment for seepage (the Design Guideline for River Levees, 2012)

Failure mode		Standard
Piping	Without covered silt or clay near the toe	i < 0.5 i; Largest of local hydraulic gradients of the foundation ground near the toe
	With covered silt or clay near the toe	G/W > 1.0 G: Weight of the covered soil W: Uplift force acting on the bottom surface of the covered soil
Slip	Back slope	Fs ≥ 1.2 × α1 × α2 Fs: Factor of safety against slip failure α1: Extra coefficient for the complexity of soil layers For complicated soil layers, α1= 1.2 For simple soil layers, α1 = 1.1 For new constructed levee, α1 = 1.0 α2: Extra coefficient for the complexity of foundation ground For piping history or topography requiring attention, α2 = 1.1 For without piping history or topography requiring attention, α2 = 1.0
	Front slope	Fs ≥ 1.0 Fs; Safety factor against slip failure

Remediation measure(s)

Remediation methods for seepage in Japan

Six remediation methods for seepage are on “the Design Guideline for River Levees”. All these measures are implemented at the sites.

1. Enlargement for piping/slope stability

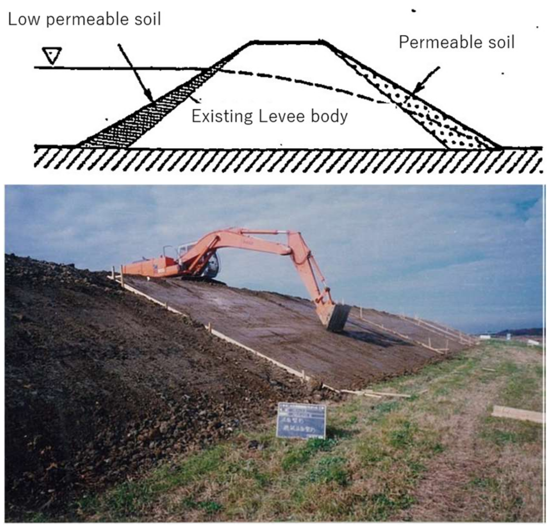

Figure 3.12 Enlargement for piping/slope stability (the Design Guideline for River Levees, 2012)

2. Drain Method for slope stability

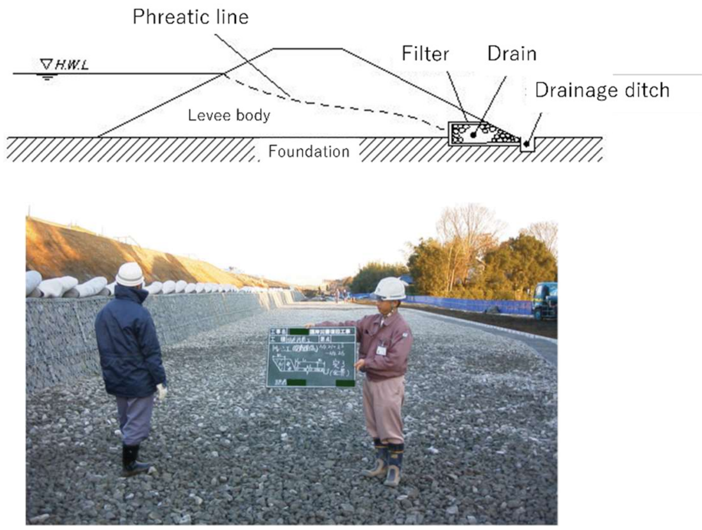

Figure 3.13 Drain method for slope stability (the Design Guideline for River Levees, 2012)

3. Impermeable Surface Layer for slope stability

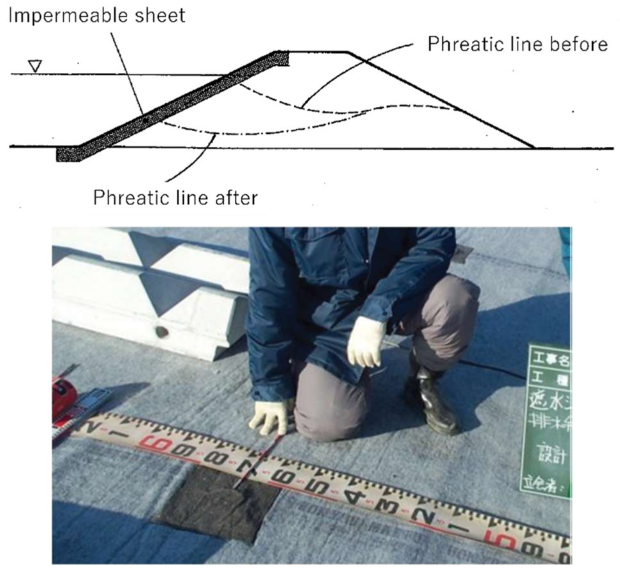

Figure 3.14 Impermeable surface layer for slope stability (the Design Guideline for River Levees, 2012)

4. Blanket Method for piping

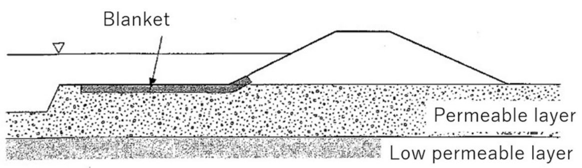

Figure 3.15 Blanket method for piping (the Design Guideline for River Levees, 2012)

5. Cutoff Wall for piping

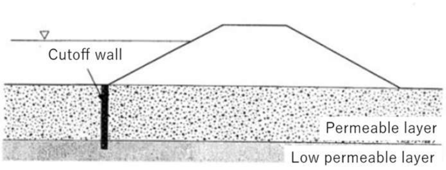

Figure 3.16 Cutoff wall for piping (the Design Guideline for River Levees, 2012)

Other information about this case

Levee maintenance in Japan

Levee maintenance in Japan consists of four components: inspection, evaluation, monitoring and repair. The inspections are carried out at least twice a year, before the flood season and the typhoon season. In addition to that, the inspections take place after a flood or an earthquake. The inspection and the result evaluation are based on “the Technical Standards for River and Sabo: Maintenance (Rivers)” by MLIT. The details are presented in “the Maintenance Guidelines for Levees and the Other River Management Facilities and River Channels” also by MLIT.

The maintenance guideline covers the inspections and evaluation conducted to maintain flood control structures in terms of ensuring that a) river channels to maintain the planned flood flow capacity and b) levees and other river management facilities have the required functions. They present standard methods about detailed inspections (including surveys and monitoring), the evaluation of inspection results to determine whether a repair is needed.

The items covered by the maintenance guidelines includes earth levees including the seepage remediation methos, revetments, seawalls, special levees with self-standing structures or parapet structures, land locks, sluiceways, flood gates, and weirs.

References

the Design Guideline for River Levees (2012 Japan Institute of Country-ology and Engineering(JICE) , Japanese)
http://www.jice.or.jp/cms/kokudo/pdf/tech/material/teibou_kouzou02.pdf
the Technical Standards for River and Sabo: Design (2023 MLIT, Japanese)
https://www.mlit.go.jp/river/shishin_guideline/gijutsu/gijutsukijunn/sekkei/index.html
the Technical Standards for River and Sabo: Maintenance (2021 MLIT, Japanese)
http://www.mlit.go.jp/river/shishin_guideline/gijutsu/gijutsukijunn/ijikanri/
the Maintenance Guidelines for Levees and the Other River Management Facilities and River Channels (2023 MLIT, Japanese)
https://www.mlit.go.jp/river/shishin_guideline/kasen/pdf/01_teibou_tenkenhyouka_youryou_r503.pdf

3.1 3 Case description: Sheet pile cut-off wall, impermeable sheet and drainage at the Yabe River Levee (internal erosion)

Authors: Hirotoshi Mori, Shunsuke Sako, Japan
Keywords: Piping; River levees; increasing hydraulic loads (intensity or frequency) due to climate change

For the corresponding technique factsheet on sheet pile walls, please refer to Section 3.2.3.

Setting

Internal erosion at the Gono River levee

There are a number of cases of levee damage due to internal erosion in Japan, including the large amount of sand-boil from the subsoil of the Gono River levee caused by the torrential rain in July 2018 in western Japan (Figure 3.17: Left). Regarding the breach caused by internal erosion of river levees, the Investigation Committee concluded that the levee breach on the Yabe River in July 2012 (Figure 3.17: Right) was caused by piping (Yabe River Levee Investigation Committee Report, 2013).

Figure 3.17 Left: Gono River levee sand boil in the Torrential rains in the western Japan of July 2018, Right: Yabe River levee breach by piping of July 2012 (MLIT)

Problem description

Internal erosion at the Yabe River levee

Figure 3.18 illustrates the process and failure mechanism of the Yabe River levee by piping. At the site, a sand layer was present beneath the levee body. The river water level remained high, and the hydraulic pressure of the river water propagated through the foundation soil. However, the land side of the levee constituted a dead end for the sand layer, and there was no place for the water pressure to escape. As a result, the pressure caused the sand-boil. Consequently, piping was initiated and continued until the settlement of the levee body, resulting in overtopping and breach of the levee.

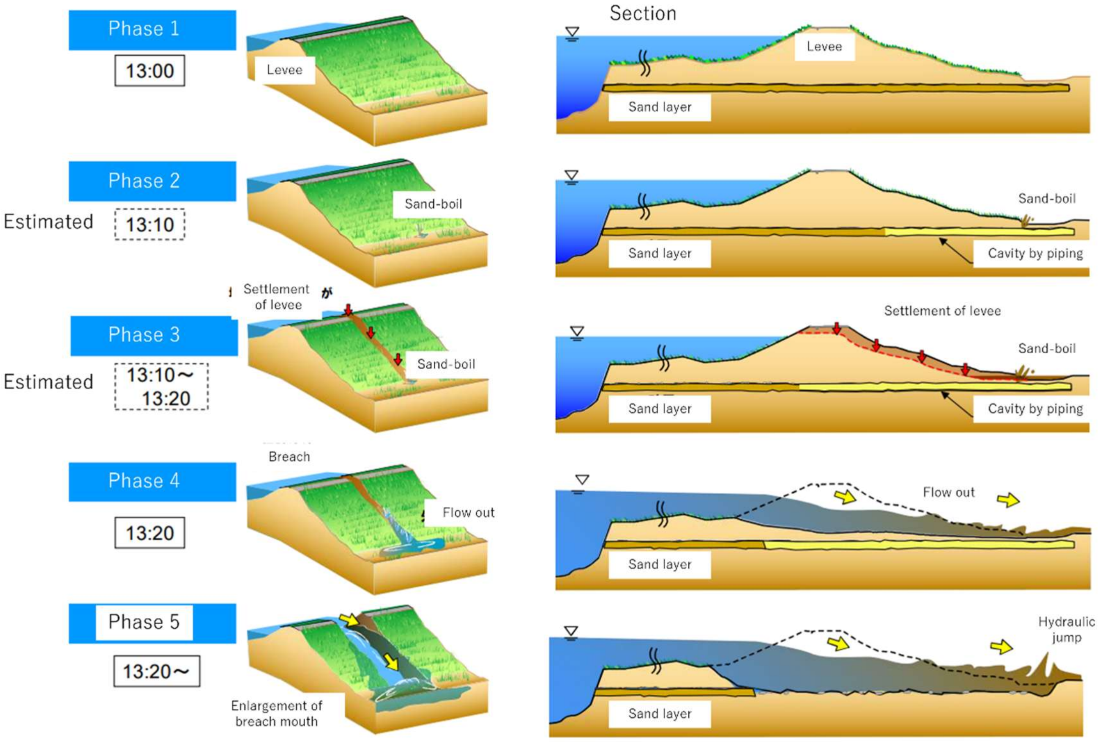

Figure 3.18 The failure mechanism of Yabe River levee breach (MLIT)

Remediation measure(s)

Cutoff wall using sheet pile at the Yabe River levee

As a means of mitigating the effects of a potential levee failure, a cutoff wall using sheet piles was planned, designed, and constructed as illustrated in Figure 3.19. The cutoff wall was installed at the toe of the river side and its depth extended up to the Acs (silty sand) layer, which has low permeability. To prevent the infiltration of river water into the levee body, impermeable sheet was also installed on the river side slope. In addition, drainage was employed on the inland side toe of the levee.

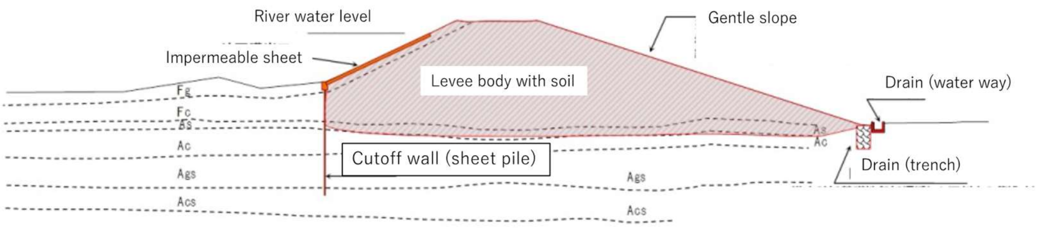

Figure 3.19 Design for Yabe River levee reinforcement including cutoff wall and drainage

Cutoff walls are considered as one of the countermeasures for piping. The unsteady saturated/unsaturated seepage analysis is performed for the design of cutoff wall with rainfall and river water level. The hydraulic conductivity of 1.0e-9 m/s for 1 mm thick sheet pile is used for the analysis, which is given in the Design Guideline for River Levees, 2012.

References

Yabe River Levee Investigation Committee Report (2013), Yabe River Levee Investigation Committee, Japanese https://www.qsr.mlit.go.jp/chikugo/site_files/file/torikumi/01-plan_course/tyosa/saisyu/houkokusyo.pdf

3.1 4 *Case description: Sheet pile wall water side along the Danube River (internal erosion)*

Authors: Edina Koch, Richard Ray (Széchenyi István University) Márton Maller, Gergely Bartal (North-Transdanubian Water Directorate), Hungary

Keywords: hydraulic failure; backward erosion piping; river levees; clay; silt; sandy silt; silty sand; gravel; increasing hydraulic loads (intensity or frequency) due to climate change.

For the corresponding technique factsheet on sheet pile walls, please refer to Section 3.2.3.

Setting

Hungary has one of the extensive river dike systems in Europe along the two major rivers (Danube and Tisza) and their tributaries. There are 12 regional Water Directorates responsible for flood protection.

The North-Transdanubian Water Directorate (ÉDUVIZIG) is a state-owned water management institution responsible for operating and maintaining facilities in Northwestern of Hungary. The directorate's main activities include flood prevention and control, drainage and mitigation of inland excess water, provision of irrigation water, and the sustainable management of surface and groundwater resources.

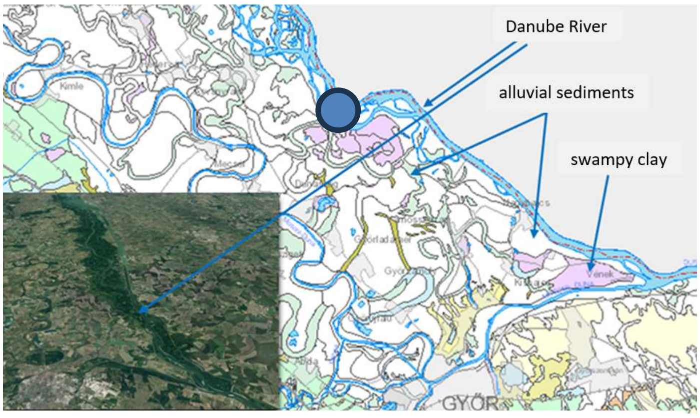

Figure 3.20 Location of the investigated case along the Danube

The protected flood-prone land covers about 1400 km^2, the primary flood defence lines extend a total of 475 km. The ÉDUVIZIG maintains a 142 km section of the Danube, including the Szigetköz floodplain and its connected main rivers: the Moson-Danube, the Marcal, the Rábca, and the Lajta.

The Szigetköz is a “hotbed” of sand boil formation due to the combination of a thick gravel layer (100’s of meters) beneath the Szigetköz covered by a relatively thin blanket of poor soil having variable thickness (Figure 3.21). Sand boils have a long tradition in this area where there are continuous technical accounts of specific boils over time (when they started and stopped, water levels, etc.). Some of them even have names.

The investigated case is a countrylike levee (Figure 3.22). The measured current height of the levee is ~4.0 m, the base width is ~35 m, and the crest is 6.0 m wide. The landside and waterside slopes of the levee is ~1:3. The foundation and core of the levee are silt and low-plasticity clay, and the bottom layer is sand and silty soils deposited by the Danube (Figure 3.23). During the last century, the levee was widened by coarse-grained material and finally covered by clay.

The typical flood event lasts about 1 week.

Figure 3.21 Sand boil

Figure 3.22 The investigated site

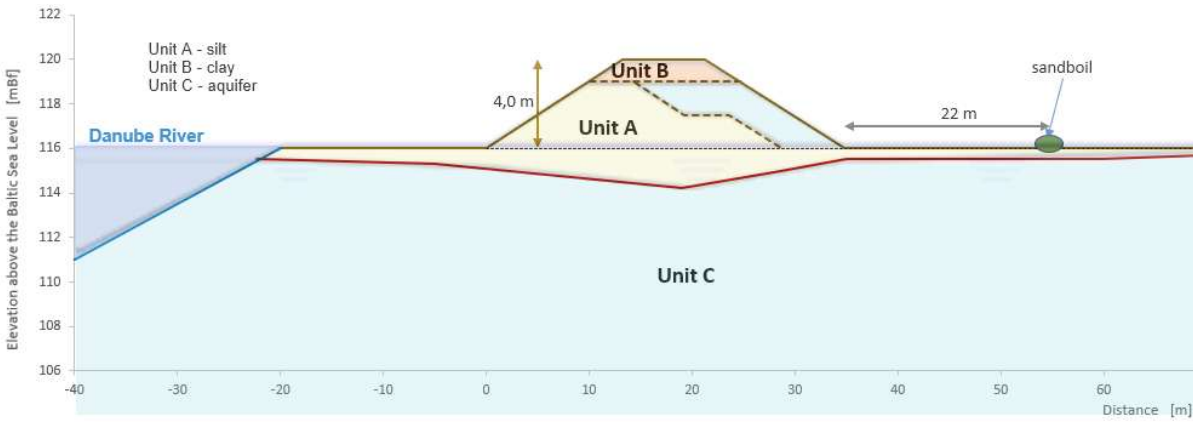

Figure 3.23 Soil profile of the investigated case

Problem description

Based on the updated safety assessment performed after the 2013 flood, some parts of the levee system do not meet the current safety requirements. The main issues are backward erosion piping, uplift, and heave. Slope stability is a minor issue. Moreover, some parts of the levee system do not satisfy the required height based on the recalculated design water level.

Remediation measure(s)

In 2013 during the largest flood event, several sand boils and sand boil groups were observed along the upper part of the Danube River. After the flood, reinforcement of the existing levee was necessary for a length of about 200 m.

There were several possibilities to reinforce the critical section e.g. deep slurry wall, sheet pile wall, jet grouting wall, filter sand berm or relief wells. These techniques are part of the common levee reinforcement techniques in Hungary. Due to the lack of space, the sheet pile wall was selected. The sheet pile wall was vibrated into the subsoil on the waterside.

Details on the sheet pile wall

The sheet pile wall was installed at the levee toe on the waterside as a mitigation strategy for the hydraulic failure. The length of the pile is 8 m. The schematic cross section is shown in Figure 3.24. Some images of the construction can be seen in the Figures below.

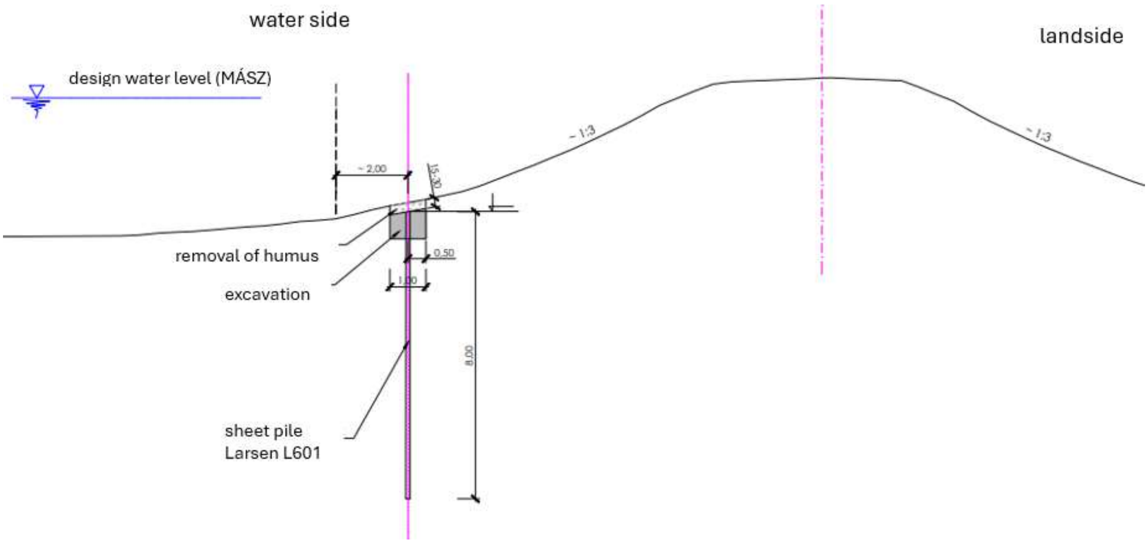

Figure 3.24 Schematic cross section

Figure 3.25 Construction of the sheet pile wall

The technique is applicable for mitigating seepage under the levee.

Performance

The levee section was reinforced in 2018. Since then, several floods (with lower flood levels than in 2013) have inundated the region, but there have been no problems with the reinforced levee. The sheet pile wall has provided an effective method for the stabilization of the subsoil and mitigation of underseepage.

Other information about this case

Due to the extreme flood event in 2013, engineers have raised the design flood parameters. The event has prompted new values for design water levels, evaluations of new levee designs, and reinforcement of existing levees.

3.1 5 *Case description: Vertical filtration & drainage in the foundation along the Alpine Rhine in Liechtenstein (internal erosion)*

Main authors: Gregor Portmann & Hansjörg Vogt (Tragweite AG, LI)
Contributors: Florin Banzer (Sprenger & Steiner AG, LI); Johannes Toepke (IUB Engineering AG, CH), Anne Christine Pfaffen (Authority for Civil Protection ABS, Principality of Liechtenstein)
Keywords: Slope sliding; internal erosion; river levees; mountain stream levees; soft clay; sand; gravel; increasing hydraulic loads due to climate change; material prices; increasing economic value in areas sensitive to flooding.

For the corresponding technique factsheet on vertical filtration and drainage, please refer to Section 3.2.4

Setting

The Principality of Liechtenstein is located between Austria and Switzerland, with the Alpine Rhine forming the border between Liechtenstein and Switzerland over a length of around 27 km (see Figure 3.26). The name Alpine Rhine comes from the fact that the Rhine only flows into Lake Constance further north, meaning that flooding is characterised by a rapid and relatively short rise in water levels, which is typical for Alpine rivers. A complete flood event usually lasts between 12 and 48 hours, with the peak discharge lasting only a few hours. The Rhine levee protects the densely populated valley plain from flooding.

Figure 3.26: The principalety of Liechtenstein with the river Rhine (blue line) (source: https://map.geo.llv.li)

The levees were built from surrounding material during the straightening of the Rhine in the 19th century. Figure 3.27 shows the typical structure of such a levee: the levee was built from locally available Rhine gravel (poorly graded gravel GP to well-graded gravel GW according to unified soil classification system USCS) with a height of up to 8.5 m. In many places, beneath the levee (or throughout the valley plain), there is a fine-grained, silty-clayey layer of flood deposits (inorganic silts ML according to USCS) of varying depth and thickness, which in turn is underlain by Rhine gravel (poorly graded gravel GP according to USCS) several metres thick.

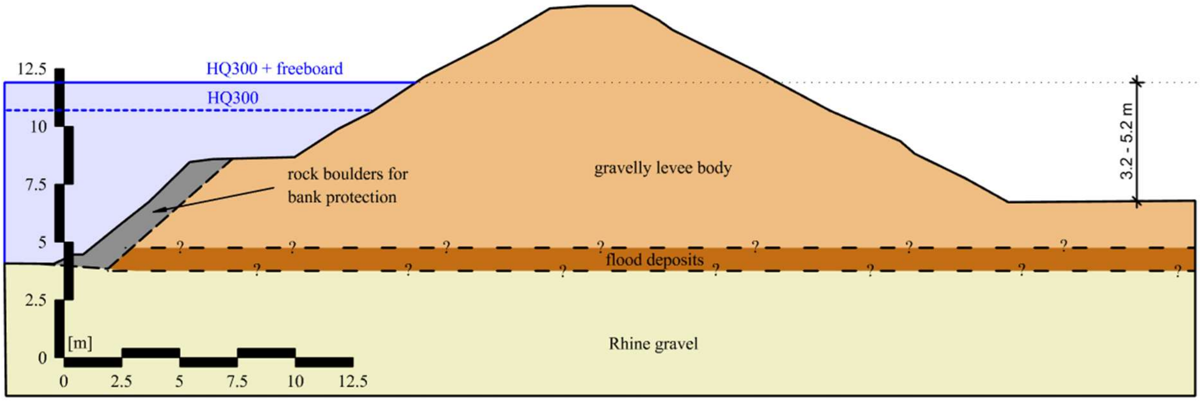

Figure 3.27: Principle geotechnical cross-section through the river Rhine levee (km 54.850)

Figure 3.27 shows the HQ 300 flood level (high discharge flood event). Figure 3.28 shows typical sieve curves of the materials, where LB stands for Levee Body, FD for Flood Deposits and RG for Rhine Gravel.

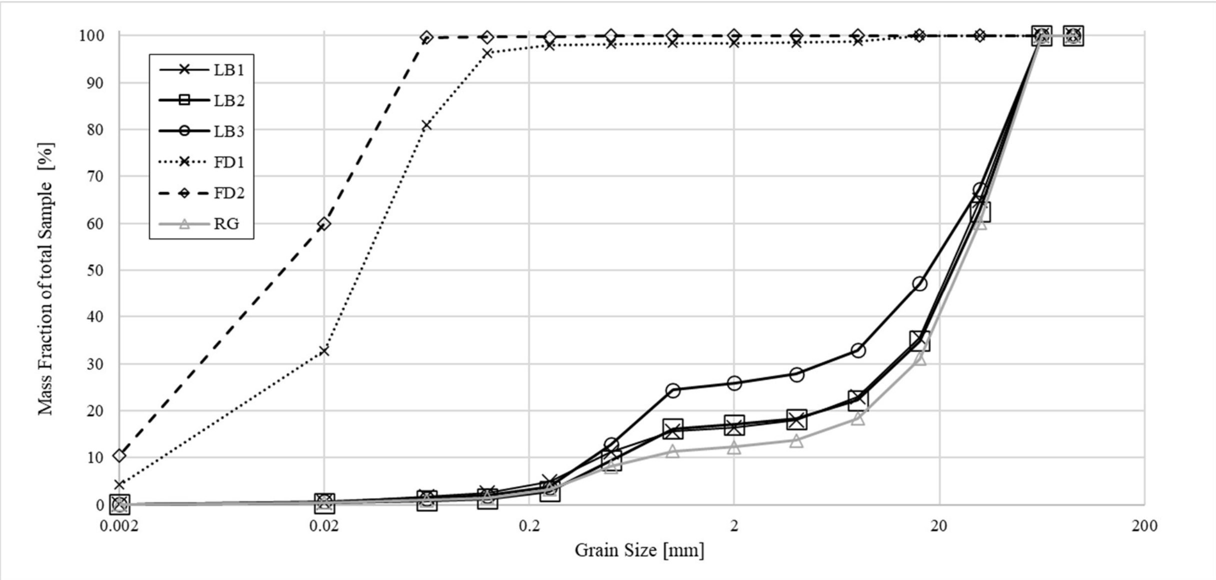

Figure 3.28: Sieve curves: LB = Levee Body; FD = Flood Deposits; RG = Rhine Gravel

This case study describes the remediation of an approximately 400 m long section in the area of a wastewater treatment plant located directly at the base of the levee (see Figure 3.29).

Figure 3.29: Remediation section directly adjacent to a wastewater treatment plant before remediation measures (yellow lines every 50 m) (source internal project documentation, Ingenieurbüro Sprenger & Steiner)

Problem description

In the course of a design review of the flood safety of the levees, it was determined that they did not provide sufficient protection against failure. The following phenomena have a negative impact on the stability of the levee:

- It cannot be ruled out that the levee body, due to its high permeability ($k_h \approx 1$ untill $5 \cdot 10^{-3}$ m/s), will become saturated and seepage will occur even during short flood events. This has a negative effect on the stability of the air-side embankment, especially if fine particles are washed out of the levee in an uncontrolled manner and progressive erosion occurs.
- It also cannot be ruled out that increased pore water pressures may develop in the area of the landside levee base below the fine graded flood deposits. On the one hand, there is a risk that uncontrolled seepage paths with progressive erosion will form (sand boils with piping); on the other hand, the increased pore water pressures destabilise the levee base (reducing the effective stresses until, in extreme cases, a slope failure can occur).

An additional complication is that a wastewater treatment plant (critical infrastructure) is located directly at the base of the levee. This means that the standard remediation measures — a land-side ballast filter and a gravel-filled trench for pressure relief — cannot be implemented here due to space limitations (see Figure 3.30a).

Remediation measure(s)

Description the selected remediation methods

Figure 3.30b shows the levee remediation carried out in winter 2024/25. This consists of the following elements:

- Gravel columns: To relieve the pore water pressure beneath the flood deposits during a flood event, filter gravel columns with an outer diameter of 900 mm are installed every 3.5 m. These are embedded around 2 m into the Rhine gravel. A two-stage filter structure is required in the area of the flood deposits to separate the filter gravel from the silty-sandy material. Therefore, a fine-grained filter gravel layer (Ø 0.06–3.6 mm, 150 mm thick) is installed around the core of coarse filter gravel (Ø 2–50 mm) with a diameter of 600 mm.

- Ballast filter: To control seepage and increase the stability of the levee, a ballast filter with a 1:2.5 slope gradient has been installed. Due to insufficient space at the base, a retaining wall with drainage openings in the slap and outlet openings in the stem has been constructed. The height of the wall is determined by the terrain geometry, with a terrain difference of between 0.5 and 2.0 metres.

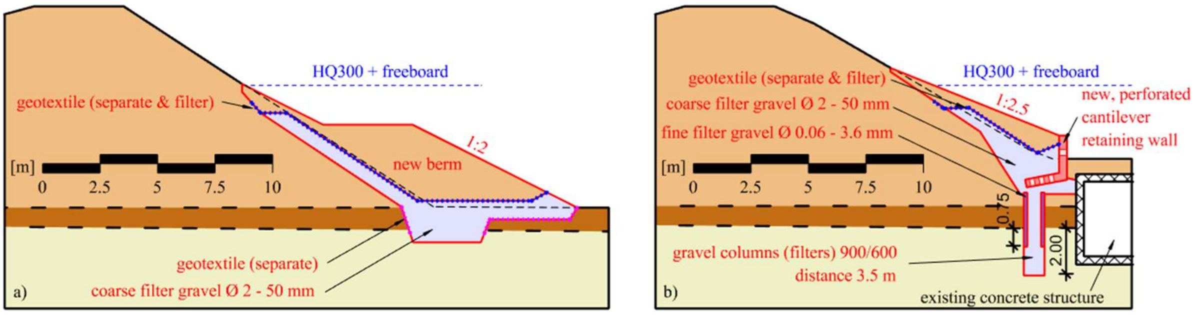

Figure 3.30 Typical cross-section of the river Rhine remediation measures: a) usually carried out restoration with ballast filter and pressure relief trench; b) completed levee reconstruction with pressure relief gravel columns, ballast filter and retaining wall.

Performance

Since the remediation of the section presented (winter 2024/2025) until the publication of this article (summer 2025), no major flooding has been recorded. Consequently, no statements can be made about the effectiveness of the measure presented during a flood event.

Other information about this case

During a flood event, monitoring measures are planned to check the effectiveness of the remediation measures. In the current remediation section, it is expected that seepage water will exit the openings in the retaining wall at approximately HQ30 (high discharge with an annual probability of occurrence of 3 percent) + freeboard. Here, inspections will be carried out to determine whether the seepage water carries large quantities of fine material – this would be an indication that the filter structure is not working as intended.

Author statement

For the sake of completeness, the roles of the individual authors involved in planning and execution should be mentioned here. Client: Liechtenstein Civil Protection Agency, represented by Anne Christine Pfaffen. Designer: Assessment of remediation measures by IUB Engineering AG (Johannes Toepke); concept and feasibility of filter gravel columns by Sprenger & Steiner Anstalt AG (Florin Banzer). Execution: Wilhelm Büchel AG (construction company). Hanno Konrad Anstalt (Benjamin Lind) as the construction manager. Tragweite AG (Gregor Portmann and Hansjörg Vogt) were responsible for supervising the construction work as geotechnical specialists.

3.1 6 *Case description: Treatment of the land-side slope with mesh against burrowing animals (internal erosion)*

Author: V. Gili

This contribution is based on: *FICHE DE CAS Confortement des digues des Amidonniers/Sept-Deniers (Digues D12, D13 et D14) Traitement du talus aval par grillage anti-fouisseurs Garonne – Toulouse (31).' From Recueil de méthodes et de techniques de confortement et réparation des digues de protection en remblai of the Comité Français des Barrages et Reservoirs (CFBR).*

Setting

Identification of the segment affected by the work.

The segments (D12, D13 and D14) are located along the river Garonne in Toulouse (managed and owned by Occitanie/Haute-Garonne) and are lined levees with a riverside face, see figure 3.31.

Figure 3.31 Project locations D12 (left) and D14 (right)

The segment lengths are: 1421 m (D12), 1011 m (D13) and 895 m (D14) respectively. The regulatory class of the structure is B, but pending the reclassification of the containment system it could become A.

Protected area:

The protected area is Amidonniers/Sept-Deniers/Ginestous, department 31. The area consist of a permanent population of 15,670 inhabitants (according to the 2015 municipal flood plan and in reference to the historic flood of 1875). Major roads are A620/A621 (but vulnerability not fully established in view of the characteristics of these high-embankment roads) and other major stakes are 5 schools, 1 wastewater treatment station, 1 hospital and business parks.

Levee characteristics

A cross-section of the dike is shown in Figure 3.32. Date of construction is 1960s and type of levee is a lined levee with a concrete face.

Height of the levee with respect to the ground surface on the protected area side: minimum height for the segment: 0 m (given that the areas of high embankment constitute the ground surface) and maximum height for the segment: approximately 6.40 m.

The slope on the watercourse side is mainly 1H/1V before work for D12 and D14; 1H/4V for D13; on the protected area side: 1.7H/1V to 1.4H/1V.

Crest width is variable from 3 to 5 m + areas with back embankment and the levee consists of sandy silt and silt-gravel sand (heterogeneous).

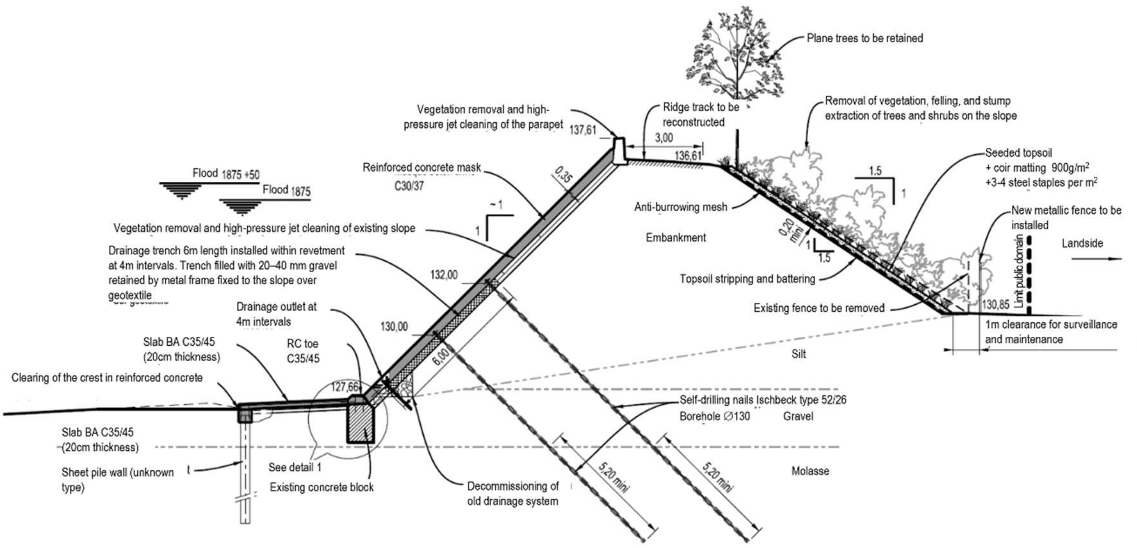

Figure 3.32 Cross-section of the dike

Table 3.4 Flood conditions

Approximate frequency	25 years (2000 flood)	100 years	Reference 300 years (1875 flood)	Exceptional 600 years (1875 flood + 50 cm)
Peak discharge	5100 m^3	5100 m^3	7500 m^3	8250 m^3
Dimension / what reference?			D12 : 137.5 to 136 m D14 :	
Water height / what reference is available?	4.38 m (Pont-Neuf station)	/	7.55 m (Pont-Neuf station)	8.05 m (Pont-Neuf station)
Overflow height (-)	D12 : 137.5 to 136 m D13 : 136 to 135.4 m D14 : 134.5 to 132.1 m excluding parapet + 1m with parapet			
Water line available			D12 : 135.8 to 134.4 m D13 : 134.4 to 133.7 m D14 : 133.5 to 132.3 m	
Hydrogram attached?	See below	See below	See below	no
Hydrological study reference(s)	CACG (1988) and BCEOM (1994) studies			
Hydraulic study reference(s)	Hydraulic studies of the Garonne in Toulouse, Sogréah (2003 and 2006)			

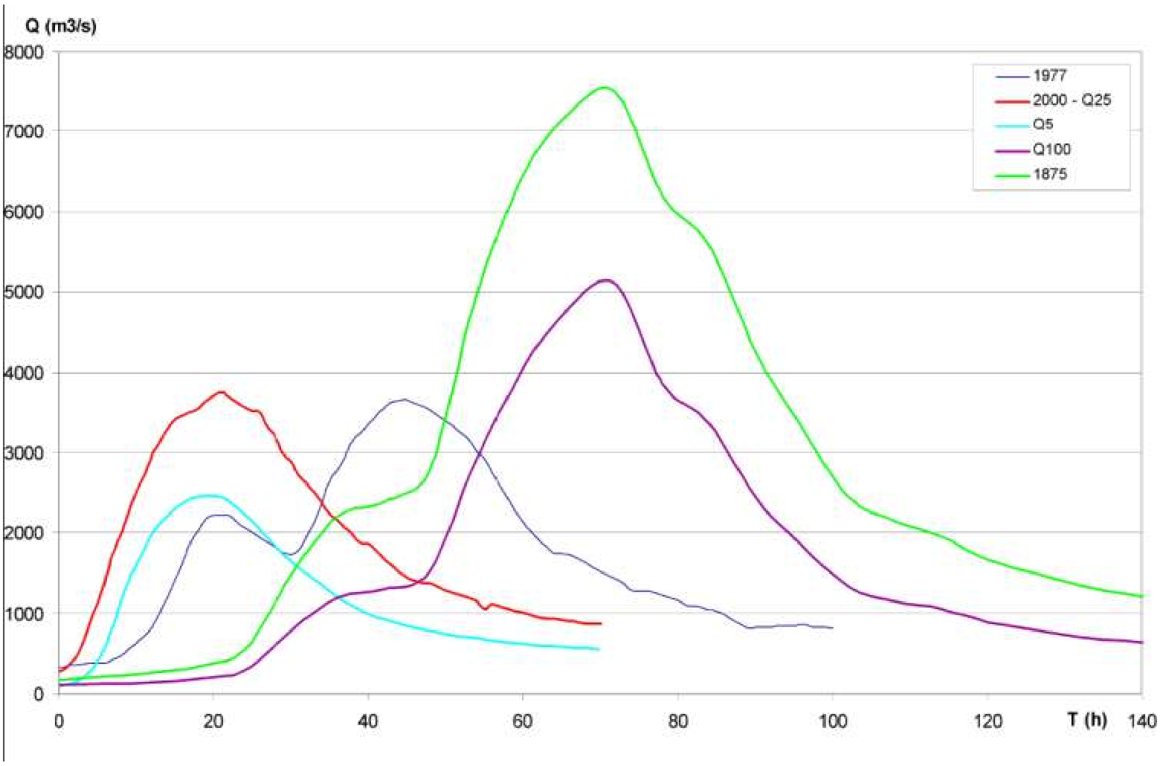

Figure 3.33 Hydrograms of the floods of 1977, 2000, 1875 and 5 years and 100 years return periods (Sogréah, March 2006)

Major earlier upgrading work was reinforcement and raising of the parapet in certain areas around 1997

Other characteristics of the levee are:

- Several areas with back embankment of several tenths of metres
- D12: a 300 m section with a retaining wall on the protected area side
- D14: an area without sheet pile on the river side

Hydraulic operation and flood conditions (see Table 3.4 and Figure 3.33) of the watercourse are:

- Discharge and frequency of the design-basis flood: 7500 m³: approximately 300 years (1875 flood)
- Flood discharge at various return periods: see next page.

Morphodynamic operation of the watercourse is:

- Distance from the levee to the watercourse: 4 m minimum (at low water)

Problem description

Damage underlying the work and/or reason for reinforcement:
On the river side was substantial and serious damage (slab splitting, etc.).
On the protected area side, numerous types of damage were observed as unwanted vegetation growth, presence of numerous burrows, suspected subsidence, etc.
Assumed degradation mechanism is a scenario of instability through internal erosion (mainly)

Remediation measures

The main objectives of the reinforcement work undertaken on D12 and D14 are:

- the construction of a new impervious concrete mask on the river side and its anchorage by nails sealed in cohesive soil (molasse)
- restoration of the embankment drainage system
- treatment of the downstream slope

The purpose of this case is to illustrate the treatment of the downstream slope, mainly with regard to the problem of burrowing animals. This case is therefore dedicated to the protection device used on the downstream slope.

Objectives of the chosen reinforcement:

- Reduce the risk of internal erosion (deep hydraulic seepage, etc.) in the slope
- Permanently facilitate monitoring and maintenance of the downstream slope
- Ensure better landscape integration of the levee (located in an urban area and a popular place for walks)
  N.B. in particular, the choice was made to keep the plane trees at the crest, to take them into account in the design of the project while at the same time putting in place phytosanitary monitoring of these trees.

Description of the work

New structures or parts of structures (I-wall, 19314 m² of mesh against burrowing animals, etc.): installation of mesh against burrowing animals.
Main changes to existing structures (changes in profile, etc.) are complete removal of slope vegetation (grubbing up, root removal), stripping by approximately 10-15 cm, adjustment of the slope to a regular slope of 1.5H/1V, addition of topsoil *(step 5)*, coconut mat and seeding.

An alternative to the installation of a mesh against burrowing animals would be the installation of a concrete face, but that was not chosen, because it was a more expensive alternative solution.

Details of the work

The following information is mainly based on the project and the tender documents drawn up by the project manager, as well as the implementation procedure drawn up by the contractor and approved by the project manager

Detailed description and step-by-step specifications are shown below and in Figure 3.34 until Figure 3.38:

- Step 1: complete removal of vegetation from the slope
  - Implementation:
  Felling and stump removal of all trees and shrubs on the levee (except the plane trees)
  Particular care is taken around the plane trees at the crest of the slope so as not to damage their root system; treatment of damaged roots
  - Comments (difficulties encountered, etc.):
  Anticipation of this step according to the specifications of the Order made under the Environmental Code
- Step 2: stripping
  - Implementation:
  Stumps removal by excavator of stumps that could not be removed in step 1
  Stripping of the slope by 10-15 cm to achieve a regular slope
  Disposal of material to approved landfill by 6x4 or 8x4 trucks

- Special specifications:
Removal of all stumps and roots of more than 1 cm in diameter
- Comments (difficulties encountered, etc.):
Possible reuse of the stripped soil in step 5 instead of landfill disposal
Depth of stripping to be adapted to initial slope condition (light stripping if the slope in good condition)
Occasional difficulty in managing access to bring in equipment and remove soil (also valid for other steps, in particular those requiring supplies)

- Step 3: adjustment of the slope to 3H/2V by reloading
  - Implementation:
  Adjustment only when the slope is steeper than the minimum of 3H/2V
  In these areas, supply of 0/80 random fill by the crest of the levee and emptying into a skip
  Grading of the material using the excavator from the toe of the levee and crest of the levee
  - Special specifications:
  Particle size distribution 0/80 mm well graduated; 10 to 15% by weight of elements less than 80 µm; minimum 25% crushed; sand equivalent between 30 and 50; Los Angeles coefficient less than 30.
  Stump holes to be closed
  Carefully compacted filler materials: minimum q5 compactness target
  - Comments (difficulties encountered, etc.):
  Access issues (see step 2)
  Risk to be controlled: surplus of materials in the lower area and non-uniform particle size distribution after unloading (the materials tend to roll downwards, especially those from the high particle size fraction)
  Bare slope: requirement to follow quickly with the next steps (risk of erosion), especially in stormy periods
- Step 4: installation of mesh against burrowing animals
  - Implementation:
  Prior approval by the project manager of the “mesh” product data sheet
  Supply of the mesh to the crest of the levee by telescopic truck
  Unrolling of the mesh, in the direction of the slope and perpendicular to the axis of the structure, using an unrolling device, at the top to the lined part of the crest, at the bottom to the toe of the slope
  Fastening of the mesh to the slope with HA 8 mm brackets.
  - Special specifications:
  50 mm x 50 mm single-twist mesh in class A galvanised metal according to standard NF EN 10244-2
  Wire with a minimum diameter of 2.7 mm
  Overlap between strips: approximately 10 cm to 20 cm.
  Fastening to the brackets: every meter on the mesh junctions; at least one per square meter in the centre of the strips
  - Comments (difficulties encountered, etc.):
  Difficulty installing the brackets in the highest areas
- Step 5: addition of topsoil
  - Implementation:
  Prior approval by the project manager of the “topsoil” product data sheet
  Supply of the soil at the crest of the levee and emptying into a skip.
  Grading of the soil using an excavator (light compaction), from the toe of the levee then the crest of the levee, with an average thickness of 20 cm.

- Special specifications:
Support soil screened at 20 mm, with a dry density of less than 1500 kg/m³ and compliant with standard NFU 44-551.

- Step 6: installation of coconut mat
  - Implementation:
  Prior approval by the project manager of the “coconut mat” product data sheet
  Supply of the coconut mat to the crest of the levee by telescopic truck. Hand unfolding and installation, at the top to the cycle track, at the bottom to the toe of the slope.
  Fastening with HA 8 mm U connectors
  - Special specifications
  Type H2M9 (density 900 g/m²; ground cover 65%; hardened fibres; Anjengo and Aratory ropes, construction by dm² 13 x 7).
  Overlap between strips: at least 30 cm; in addition, the upstream strips overlap the downstream strips.
  Fastening to the brackets: every 50 cm on mat junctions; at least one per square meter in the centre of the strips
  - Comments (difficulties encountered, etc.):
  Care should be taken when laying the mat so as not to jeopardise the effectiveness of step 7
- Step 7: seeding
  - Implementation
  Prior approval by the project manager of the “seed mix” product data sheet
  Hydroseeding from the crest of the levee
  - Special specifications
  Seeding density: 30 g/m².
  - Comments (difficulties encountered, etc.):
  Ineffective seeding in some areas and uneven growth: implementation of a 2nd seeding
  In summer, plan to water the slopes and repeat this operation (twice a week)

Figure 3.34 stripping of the slope (courtesy: GTM; DOE)

Figure 3.35 reloading of the slope and adjustment (courtesy: GTM; DOE)

Figure 3.36 installation of mesh against burrowing animals (courtesy: GTM)

Figure 3.37 addition of topsoil (courtesy: GTM)

Figure 3.38 after installation of the coconut mat and seeding (courtesy: DDT31)

Other information about this case

Feedback on ageing of the structure: ensure proper anchorage of the mesh and its durable coverage (high risk of vandalism in urban areas) and if possible, take the best sowing period into account when scheduling the work

In the work phase, closure of the crest of the levee in particular, a popular place for walks and a structuring axis for soft modes of transport; any other solution would have had the same impact. Reaction of the population was a favourable reception, mainly linked to reopening of the walking areas.

3.1 7 Case description: Levee Failures and Reinforcement Strategies in the Republic of Korea (internal/external erosion/slope stability)

Authors: Ilhan Chang, Saebom Kim, Ajou University, Republic of Korea
Keywords: river embankment failure; overtopping; piping; levee stability; seepage control; reinforcement

For the corresponding factsheet regarding Xanthan gum biopolymer-based soil treatment, please refer to Section 3.2.5.

Setting

Design Standards for Levees in the Republic of Korea (KDS 51 50 05)
In Korea, levee design must comply with the following standards:

Planning Considerations

Leaves must be designed to prevent flooding during events up to the design flood discharge, as specified by the flood control planning scale (target design frequency) outlined in Table 3.5 of KDS 51 14 15 (Flood Defense Criteria).
They must also remain stable against erosion, seepage, sliding, and settlement.

Design Requirements

The levee design must account for river channel and floodplain conditions, socio-economic factors, environmental considerations, embankment materials, and foundation soil properties.
Based on these conditions, the levee cross-section should be determined, followed by stability calculations to ensure safety factors are met. The finalized cross-section should reflect these results.

Stability Measures

If stability cannot be achieved—such as when riverbed materials are used countermeasures must be introduced or the cross-section design adjusted to secure safety.

Like other civil structures, levee design must include hydraulic and geotechnical stability evaluations to define suitable reinforcement measures against erosion, internal seepage, and slope failure.

Design Flood (m^3/s)	Freeboard (m)
x ＜ 200	At least 0.6
200 ≤ x ＜ 500	At least 0.8
500 ≤ x ＜ 2,000	At least 1.0
2,000 ≤ x ＜ 5,000	At least 1.2
5000 ≤ x ＜ 10,000	At least 1.5
x ≥ 10,000	At least 2.0

Table 3.5 Freeboard according to the Design Flood Discharge [1]

Freeboard in Levee Design

The **Freeboard** is the extra height added to a levee as a safety margin to account for uncertainties in river conditions and to ensure the secure passage of the Flood Water Level (F.W.L.). The values shown in Table 3.5 are not calculated precisely but are instead based on practical experience.

When considering Freeboard, the following factors must be considered:

1. Safety Factors
    - Ongoing maintenance of the levee
    - Uncertainty in flood discharge measurements
    - Uncertainty in the river's conveyance capacity
2. Riverbed and Geotechnical Conditions
    - Sedimentation within the channel
    - Settlement of the foundation ground

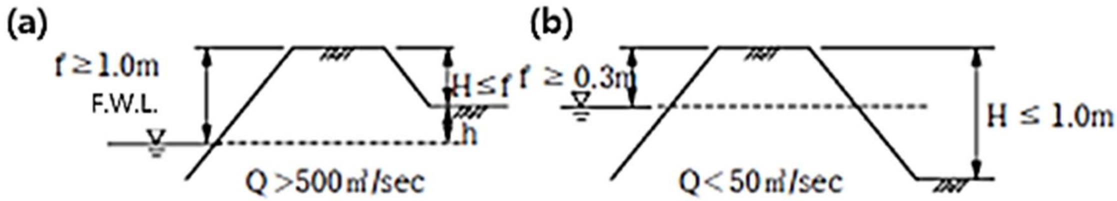

Figure 3.39 (a) Freeboard for Excavated Channels; (b) Freeboard for Small-Scale Rivers [1]

Flooding and levee failures in the Nakdong river basin during the 2002 typhoon

The Nakdong River Basin, situated in southeastern Korea, is the country's second-largest river basin. It is bordered by the Han River Basin to the north, the Geum and Seomjin River Basins to the west—where comprehensive flood control and river improvement assessments have been applied—and the Taebaek Mountain Range to the east, which separates it from the East Coast Basin.

On August 6, 2002, Typhoon Maemi brought extremely localized and intense rainfall to the Nakdong River Basin. The storm caused widespread flooding, resulting in severe damage across the region and leading to multiple instances of levee failure.

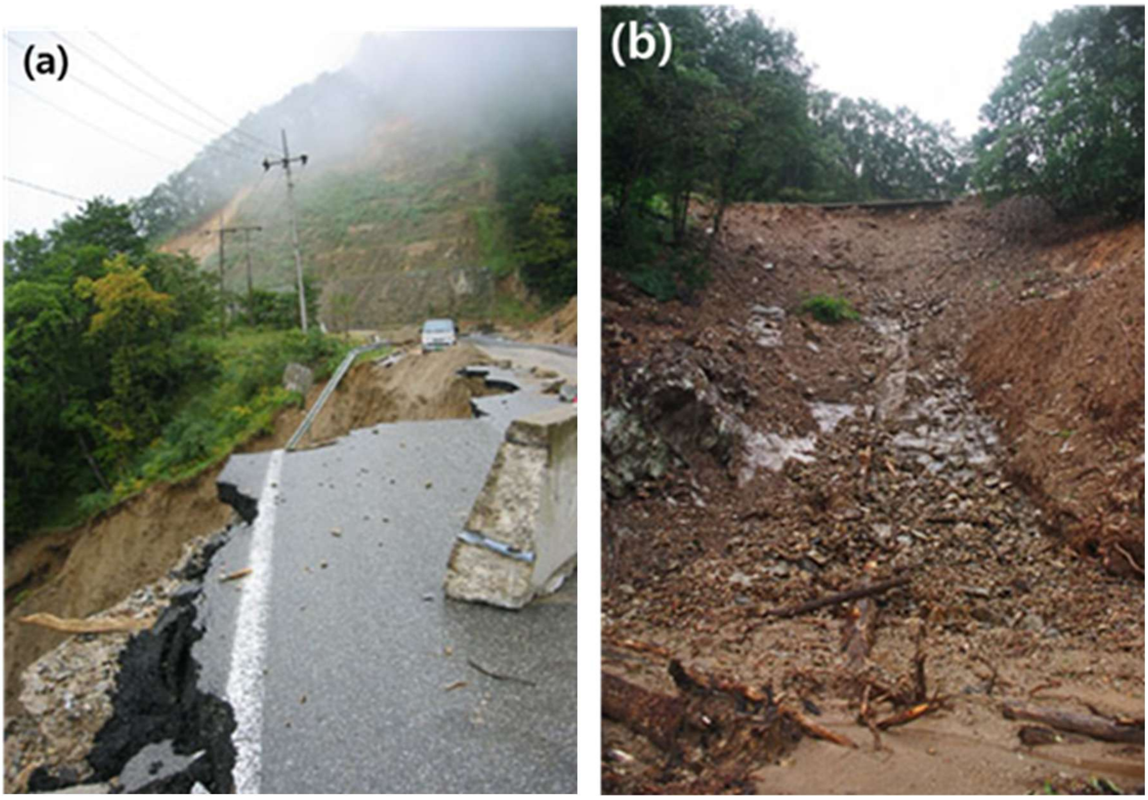

Figure 3.40 (a) Slope Collapse on the Yeongdong Expressway; (b) Collapse Situation at Oksipcheon Stream, Samcheok [2]

Levee Failure in the Seomjin River (2020)

The Seomjin River, originating from Demisaem in the valleys of Palgong Mountain in Jinan, Jeollabuk-do, flows southeastward before emptying into Gwangyang Bay at the boundary of Gwangyang-si (Jeollanam-do) and Hadong-gun (Gyeongsangnam-do). In early August 2020, prolonged and intense rainfall caused a severe flood event in southern Korea. Along the river's main stream, a levee breach of roughly 100 meters occurred, resulting in widespread inundation and the displacement of many residents.

Levee Failure in the Geum River (2023)

The Geum River, which rises near Sinmu Mountain in Jangsu, Jeollabuk-do, flows through Daejeon and Sejong before reaching the Yellow Sea between Seocheon (Chungcheongnam-do) and Gunsan (Jeollabuk-do). Key hydraulic facilities along its course include the Yongdang Dam, the Daecheong Dam, and the Geum River Estuary Bank, which play vital roles in flood control, water supply, and navigation. In July 2023, record-breaking rainfall struck central and Honam regions, rapidly elevating water levels in both the Geum River and its tributaries. This led to a partial levee failure along Sanbukcheon, a tributary in Iksan, Jeollabuk-do, causing extensive flooding and severe damage.

Figure 3.41 (a) Collapsed levee near Bridge Chungnam-si [3]; (b) Washed-out levee along the Geum River and Nonsancheon. [4]

Levee Failures in the Yeongsan River (2020)

The Yeongsan River flows through Gwangju and the western and southern parts of Jeollanam-do, creating fertile alluvial plains such as the Naju Plain. Historically, its estuary at Yeongsanpo functioned as a key port, while the lower river and coastal zone are shaped by tidal forces and currents.

In August 2020, consecutive episodes of intense rainfall across Gwangju and Jeollanam-do caused prolonged high water levels along the lower Yeongsan River. This led to partial levee failures at multiple sites, including sections along Yeongamcheon and in Hampyeong-gun. Reports described the damage as “three or four sections of the levee burst (partially destroyed).”

Figure 3.42 Levee failure along Munpyeongcheon, a tributary of the Yeongsan River. [5]

Problem Description

Reasons for Levee Repair and Reinforcement

Table 3.6 Causes of flood damage in domestic levees, Korea.

River	Time / Location	Primary Causes
Nakdong River	August 2002, Haman (Baeksan levee, Hapcheon Gwan gam levee, Gahyeon levee, etc.)	Piping and leakage at drainage culvert and pumping station joints; overtopping during heavy rainfall resulting in levee failure
Seomjin River	August 2020, Namwon (Geumgokgyo, Gurye, etc.)	Levee washout caused by overtopping; hydraulic bottleneck at bridge–levee junctions; and flood damage exacerbated by limited flood control capacity and delayed operation of the Seomjin River Dam
Geum River	July 2023, Geum River basin (Nonsan, Sejong, Buyeo areas, etc.)	Sections of levees along the Geum River were determined to have failed mainly due to overtopping-induced scour and washout, as well as piping (seepage) occurring at structure–levee interfaces.
Yeongsan River	August 2020, Gwangju and Jeonnam area (Yeongamcheon, Hampyeong, etc.)	Rapid water level rise caused by extreme rainfall; gates unable to withstand hydraulic pressure resulting in damage and backflow; partial breaching of levees

Assessment for Identifying Relevant Failure Mechanisms and Pathways

1. Flood Inundation Analysis Following Nakdong River Levee Failure

A flood inundation analysis model was developed using two-dimensional shallow water equations, designed to reflect the physical behavior of flood waves entering low-lying protected areas following levee overtopping and breach. The model's reliability and practical applicability were assessed by applying it to the case of the Nakdong River levee failure.

Figure 3.43 presents the results of a one-dimensional dynamic flood routing analysis for the river reach between Hyeonpung (upstream boundary) and Jeokpyo Bridge (downstream boundary). The figure depicts riverbed elevations and maximum flood water levels along the longitudinal profile.

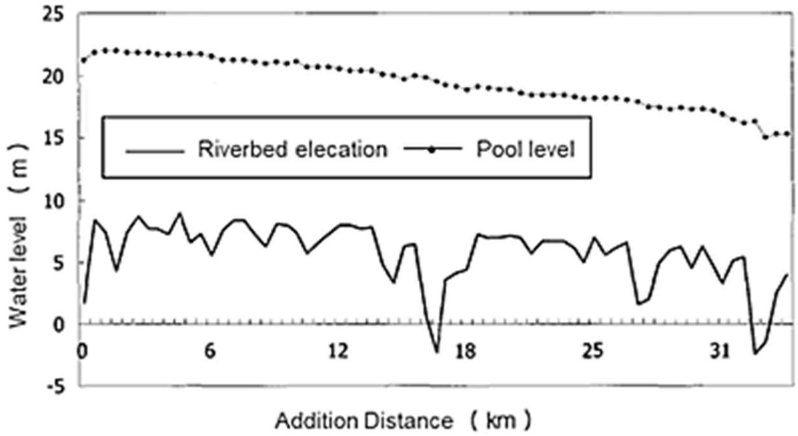

Figure 3.43 Results of flood water level analysis in the river channel. [6]

Flood inflow volume was controlled by the relative water levels of the Nakdong River main channel and the adjacent Bongsan levee-protected lowland. Inundation began at 07:40 on September 15, with discharge rising sharply until 08:30. Thereafter, flood volume gradually decreased as the main channel water level fell and the water level within the protected lowland rose.

2. Analysis of Nakdong River Levees through the Application of a Levee Failure Simulation Model [7]

A study was carried out on the Nakdong River levees using a levee failure model developed to evaluate flood discharge under erosion-based mechanisms. The model was applied to estimate the temporal evolution of breach width, the extent and depth of inundation, and the potential scale of flood damage, while also accounting for variations in river discharge throughout the failure process.

The levee failure analysis framework was structured to simulate the breaching sequence of soil-constructed levees or dams. As shown in Figure 3.44 along cross-section A–A, erosion begins at the downstream slope of the levee. With time, the erosion channel gradually deepens and extends further into the slope. The resulting discharge through this channel was computed using the broad-crested weir equation, expressed in Eq. (1).

$$Q_b = 3B_o(H - H_c)^{1.5} \qquad \textit{Eq.(1)}$$

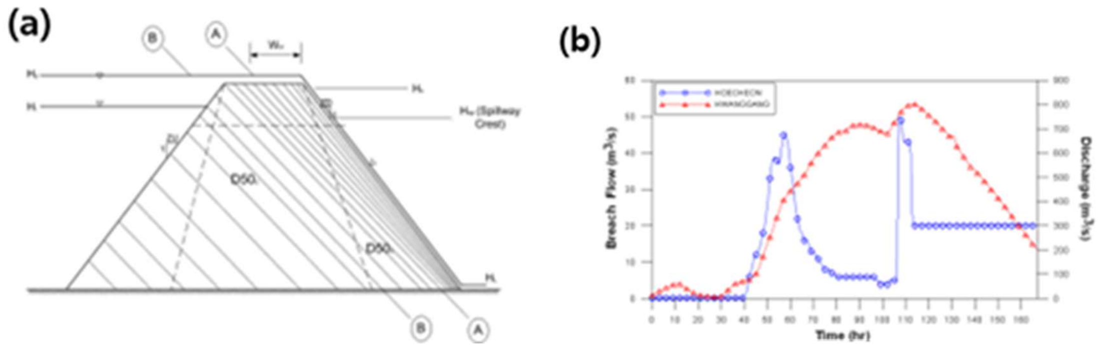

Figure 3.44 (a) Side view of dam illustrating the overtopping failure sequence [7]; (b) lateral inflow hydrograph in Hoecheon and Hwanggang.[7]

Remediation Measures

1. Commonly Applied Methods of Levee Repair and Reinforcement in Korea

The following are levee maintenance and reinforcement methods that have been traditionally and consistently applied in Korea.

Emergency Levee Reinforcement Measures
If timely and appropriate emergency measures and countermeasures are not implemented following levee failure, the risk to the structural stability of the levee can increase significantly under emergency conditions. Therefore, by analyzing past flood events and establishing suitable emergency response measures and mitigation strategies for future flood scenarios, rapid and effective response and damage reduction can be achieved.[8]

1) Soil and Rock Placement

Figure 3.45 (a) Levee protected with a temporary floodwall system [8]; (b) Examples of Stone Masonry Construction Methods [8].

2) Emergency Countermeasures Against Internal Erosion (Piping)
- Construction of Anti-Seepage Weirs

Figure 3.46 Example of anti-seepage weir installation. [8]

3) Emergency Countermeasures Against Overtopping – Use of Geosynthetics

Figure 3.47 (a) Installation process of geosynthetics [8]; (b) Example after completed installation.[8]

Permanent Reinforcement Measures

The following technologies are permanent reinforcement measures applied during normal conditions to ensure the long-term structural stability and hydraulic integrity of levees. They are preventive strengthening strategies intended to suppress potential failure mechanisms such as seepage, scour, and slope instability, while continuously maintaining and improving the safety factor and durability of the levee system.

1) Installation of Cut off Walls

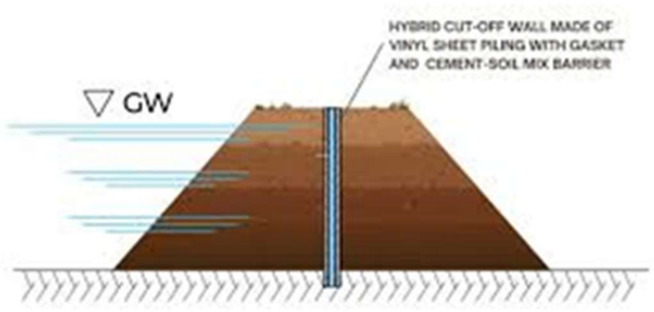

Figure 3.48 Example of cutoff wall installation method.[9]

2) Slope Protection and Installation of Erosion Control Blankets

Figure 3.49 Erosion control blanket [10]

3) Surcharge Method

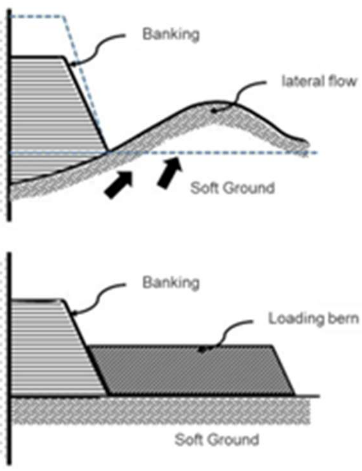

Figure 3.50 Schematic illustration of surcharge. [11]

2. Emerging Innovative Methods of Levee Repair and Reinforcement in Korea

Emergency Measures

1) Mobile Temporary Auxiliary Dams

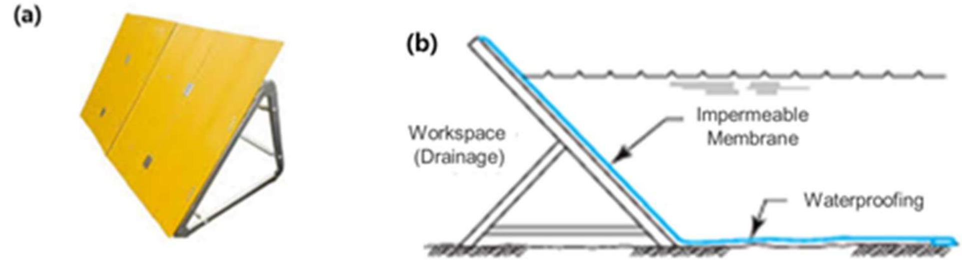

Figure 3.51 (a) Example of a mobile temporary auxiliary dam [12]; (b) mobile auxiliary dam system. [13]

Figure 3.52 Example of a water-filled tube.[8]

Permanent Reinforcement Measures

Their applicability is also increasing in terms of reducing environmental impacts, improving ease of maintenance, and ensuring long-term stability. In recent years, in response to the growing frequency of extreme rainfall and flooding driven by climate change, there has been an expanding trend toward adopting preventive and proactive reinforcement strategies based on new technologies, moving beyond conventional structure-centred approaches.

1) Levee Surface Reinforcement Using Biopolymer-based soil treatment (BPST) [14]

Figure 3.53 Field-scale application of xanthan gum biopolymer–based soil treatment.[14]

2) Enhancing Levee Stability through Composite PP-based Plastic Cut-off Walls

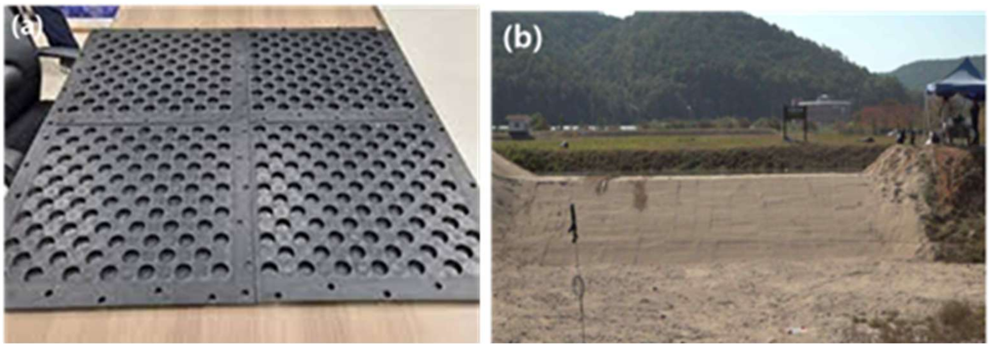

Figure 3.54 (a) Composite PP-based plastic cutoff wall [15]; (b) Example of a levee with installed composite PP cutoff walls for leakage prevention. [16]

3) Eco-Friendly Blocks for River Levee Structures

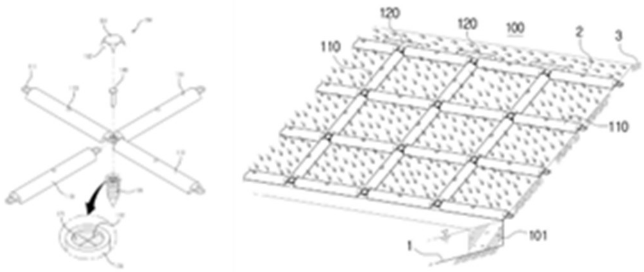

Figure 3.55 Example of a river levee structure constructed with eco-friendly blocks. [17]

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3.18 *Case description: Mixed slope protection techniques as part as a soil remediation measure (external erosion)*

Authors: Norma Patricia López Acosta, Viviana Cruz Méndez and Marco Polo Robaldi Vazquez, Instituto de Ingeniería, UNAM, Mexico

Keywords: Water flow, solute flow, plume of contamination, heavy metals, environmental geotechnics, numerical modeling.

Setting

The upper course of the Lerma River is an example of river pollution in west-central Mexico. The river faces challenges from discharges of over 500,000 residents and more than 500 industries (chemical, textile, pharmaceutical, plastic, food, automotive) in the Toluca-Lerma corridor (Guevara *et al.*, 2014).

Studying the transport of pollutants, from the river toward the adjacent lands, due to the difference in level is relevant due to their proximity to neighborhoods and farmlands. Transport of soluble pollutants via subsurface flow represents a potential risk to health and the environment due to its adverse effects and toxicity. Figure 3.56 shows the study site, including settlements and peri-urban farmland near the studied levee and the Lerma River. The detail of neighborhoods and farmlands adjacent to the Lerma River, including rural farming in areas near the levee and presence of cattle can be observed in Figure 3.57 to Figure 3.60.

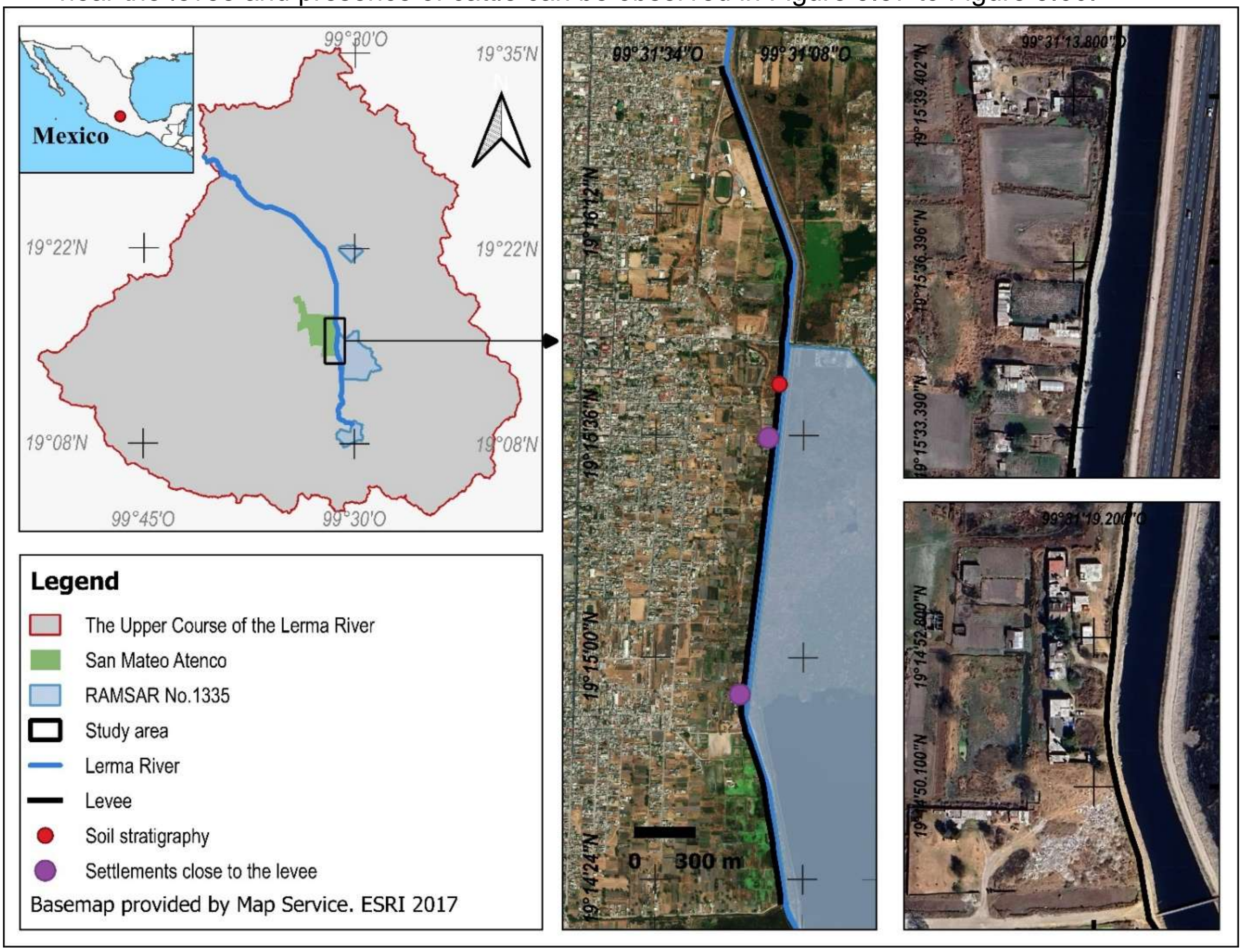


Figure 3.56 Study site. Settlements and peri-urban farmland near the left protection levee and the Lerma River

Figure 3.57 Neighborhoods and farmlands adjacent to the Lerma River

a) b) c)

Figure 3.58 Excavation for irrigation in adjacent areas to the levee. Source: a) Robaldi-Vazquez (2023a). b, c) Jiménez Gonzáles (2019)

a) b) c)

Figure 3.59 Identification of construction and settlements close to the levee. Source: a) Robaldi-Vazquez (2023b). b, c) Jiménez Gonzáles (2019)

Figure 3.60 Rural farming in areas near the levee and presence of cattle. Source: a) Robaldi-Vazquez (2023c)

Problem description

3.8 km of levees along the Lerma River in San Mateo Atenco avoid flooding and direct pollution from the river. However, the topographical difference between the water elevation in the Lerma River and the adjacent plain areas creates a hydraulic gradient and water flow in the subsurface under the plain areas, which transports soluble pollutants (see Figure 3.61). In this scenario, crops are exposed to absorb contaminants via roots and potentially affect the food chain.
An important contribution of this study is that currently no pollutant transport analysis related to the Lerma River and its banks has been conducted at the site of interest.

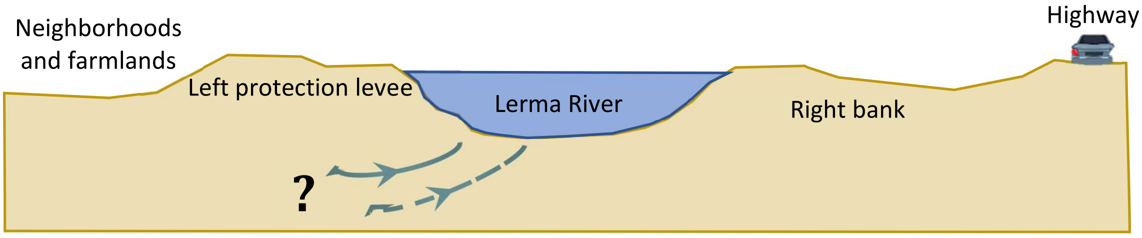


Figure 3.61 Graphical representation of the problem (Cruz-Méndez, 2024)

Scope and pollution severity

The study of pollutant transport allows us to know how the concentration of the pollutant varies over time and at different points in the region of interest, i.e., it allows us to determine the distances of pollution propagation along the river bank and to consider them to establish or propose warning and risk prevention measures for the inhabitants of the area, as well as to propose soil remediation measures.

Solute transport

Solute transport analyzes the movement of substances from the discharge zone to its surroundings, where water is the main factor that favors or conditions pollutant transport.

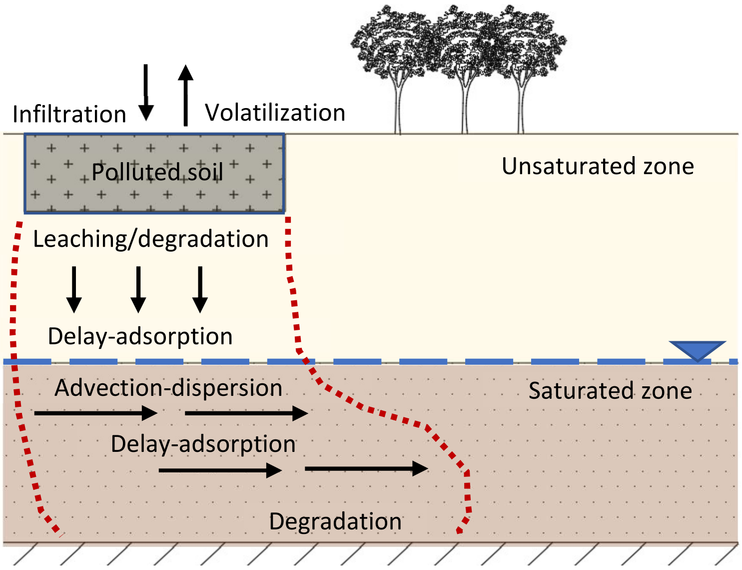


Figure 3.62 Processes that affect the movement of contaminants in the subsurface (Charbeneau & Daniel, 1993)

In the unsaturated zone, when pollution infiltrates the subsoil, it can move deeper by leaching, degradation, or physicochemical processes. This displacement occurs vertically. When the pollutant comes into contact with water, it moves depending on the direction and velocity of the water, this type of transport is known as advection. However, soil particle sizes cause the movement of the contaminant to be non-linear and to occur in a tortuous manner, which is called *dispersion*. In addition to the physical phenomena mentioned above, there are chemical or biochemical phenomena that modify the destination of contaminants transported in groundwater. When the movement of the pollutant is impeded, it is known as retardation and is a reversible process. If the pollutant adheres to the soil and is thereby displaced, the transport process is known as adsorption. Figure 3.62 illustrates the processes that affect the movement of pollutants in the subsurface.

Analytical approach of pollutant transport

The general pollutant transport equation for saturated zones, in a homogeneous, isotropic, one-dimensional medium is as follows (as cited by Iturbe Arguelles, 2014):

$$\frac{\partial C}{\partial t} = \underbrace{D\frac{\partial^2 C}{\partial x^2}}_{\text{(dispersion)}} \underbrace{- V_s\frac{\partial C}{\partial x}}_{\text{(advection)}} \underbrace{- \frac{\rho_b}{n}\frac{\partial C_s}{\partial t}}_{\text{(adsorption)}} \underbrace{+ \left(\frac{\partial C}{\partial t}\right)_r}_{\text{(degradation)}} \quad (1)$$

where *D* is the hydrodynamic dispersion coefficient, V_s the average linear seepage velocity of the groundwater, ρ_b is the bulk density of the soil, *n* is the porosity, *C* is the concentration of the contaminant, C_s is the adsorbed concentration, *t* is the time and $\left(\frac{\partial C}{\partial t}\right)_r$ corresponds to the term for degradation.

The equation comprises the transport processes in saturated zones: dispersion, advection, adsorption and degradation. *Dispersion* refers to the apparent mixing and diffusion of the pollutant within the flow system. *Advection* refers to the movement of the pollutant with the water flow. *Adsorption* is a chemical process involving the soil-pollutant reaction. *Degradation* is the lifetime of the pollutant.

On the other hand, the dispersion can be longitudinal or transversal, to obtain it there is a laboratory test, in which a tracer is passed through an unaltered soil sample, contaminant concentration variation curves are obtained, and at the end an adjustment is made to obtain the dispersion value. The infiltration rate is obtained using Darcy's equation, which relates soil permeability, hydraulic gradient, and soil porosity. Degradation depends on the type of contaminant and is measured as a function of time.

Numerical methods

A solute transport model is performed semi-coupled with a water flow model. First the water flow model is carried out to establish the subsurface flow conditions and their evolution with time, then with the solute transport analysis calculations are performed according to the configurations that have been established for evaluation. Solute transport in soil can be modeled by approximating the general contaminant transport equation, which is solved with numerical methods that employ finite element or finite difference methods, among which the following stand out: CTRAN/W and MODFLOW.

CTRAN/W is a module of the Geostudio software that uses the results of water flow analyses performed with SEEP/W (another Geostudio software module) to model the movement of contaminants through porous media. In CTRAN/W, particle movement and advection-dispersion analyses are performed. On the other hand, Visual MODFLOW allows to design and optimize pumping well locations for mine dewatering projects, but also with this software it is possible to determine contaminant fate and exposure pathways for risk assessment.

Case description

The following case study shows the relevance of analyzing solute transport in the subsurface. Figure 3.63 presents geotechnical information collected *in situ*.

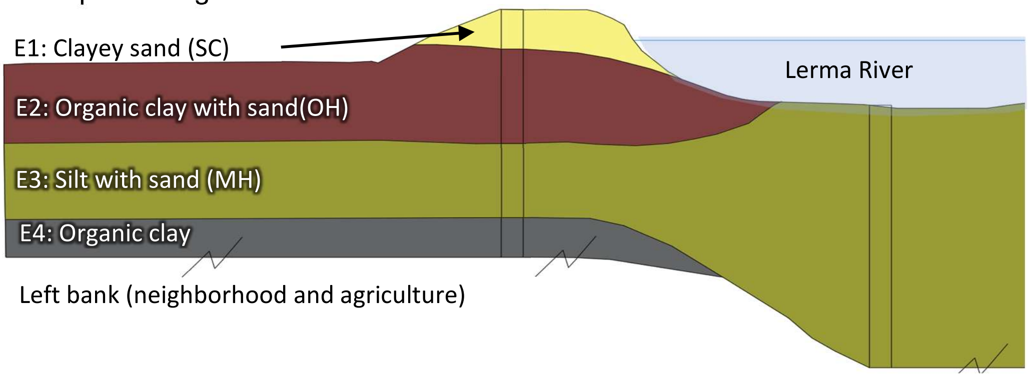


Figure 3.63 Soil stratigraphy at the study site, located at 19°15'44.32"N and 99°31'12.58"O.

For the evaluation of soluble pollutant transport at the study site, GeoStudio software is used, specifically its modules SEEP/W for water flow analyses and CTRAN/W for solute flow analyses. The parameters used for the water flow analysis are presented in Table 3.7.

Table 3.7 Geotechnical parameters for water flow analysis.

Strata	SUCS	Description	Saturated permeability, K_{sat} (m/s)	Minimum water content, w_{min} (%)	Specific gravity, G_s	Void ratio, $e = wG_s$	Volumetric water content, $\theta_w = (e/(1+e))S$
E1	SC	Clayey sand	1.00 x 10^{-4}	16.50	2.66	0.44	0.31
E2	OH	Organic clay with sand	1.00 x 10^{-7}	81.90	2.60	2.13	0.68
E3	MH	Silt with sand	1.00 x 10^{-6}	84.50	2.73	2.31	0.70
E4	OH	Organic clay	1.00 x 10^{-8}	111.50	2.60	2.90	0.74

Note: *S* = Degree of saturation = 1 (saturated medium), SUCS = Unified Soil Classification System.

Several pollutants are in the Lerma River, but heavy metals are highlighted mainly because of their harmful effects on humans, animals, and ecosystems (Kabata-Pendias & Mukherjee, 2007). The soluble lead (Pb) concentration of 1.0 g m^{-3} taken at the river's reach adjacent to San Mateo Atenco and reported by Oca-Jiménez *et al.* (2022) was selected to assess the contaminant's spread over 50 years and compare it with the reference concentrations (permissible limits) in Mexican laws (see Table 3.8).

Table 3.8 Reference concentrations (permissible limits) for lead (Pb) in Mexican laws

NOM-127-SSA1-2021(1)	NOM-001-SEMARNAT-2021(2)	NOM-147-SEMARNAT/SSA1-2024(3)
Maximum concentration of Pb in water for human and irrigation	Maximum concentration of Pb in wastewater discharges in rivers	Pb concentration to determine if soil is polluted
0.01 g m^{-3}	0.4 g m^{-3}	0.5 g m^{-3}

Source: (1)SSA1 (2022); (2)SEMARNAT (2022); (3)SEMARNAT/SSA1 (2007).

For the solute flow analyses, a lead diffusion coefficient of D=7.36x10^{-10} m^2 s^{-1} was considered (Roque-Martínez, 2015). Using the formula $\rho_d=(G_s/(1+e))\rho_w$, the density values of each stratum were obtained. It was considered that soluble lead does not present decomposition. The dispersion relation was established as shown in Table 3.9. The distribution coefficient K_d was assumed to be equal to 1 (Allan-Freeze & Cherry, 1979; Fetter *et al.*, 2018).

Table 3.9 Dispersion values used for solute flow analysis

Strata	SUCS	Description	Longitudinal dispersion, α_L (m)	Transverse dispersion, α_T (m)	Dispersion ratio, α_L/α_T
E1	SC	Clayey sand	4	2	2
E2	OH	Organic clay with sand	1	0.5	2
E3	MH	Silt with sand	2	1	2
E4	OH	Organic clay	1	0.5	2

Note: SUCS = Unified Soil Classification System.

With the above information, numerical modeling is performed, and the plume of contamination shown in Figure 3.64 and Figure 3.65 are obtained, for periods of 5 and 20 years, respectively. In the previous figures it is observed that the pollution moves in higher concentration in the more permeable strata. In 20 years, the pollution may extend 10 m on the surface of the left bank (distance considered from the toe of the embankment slope).

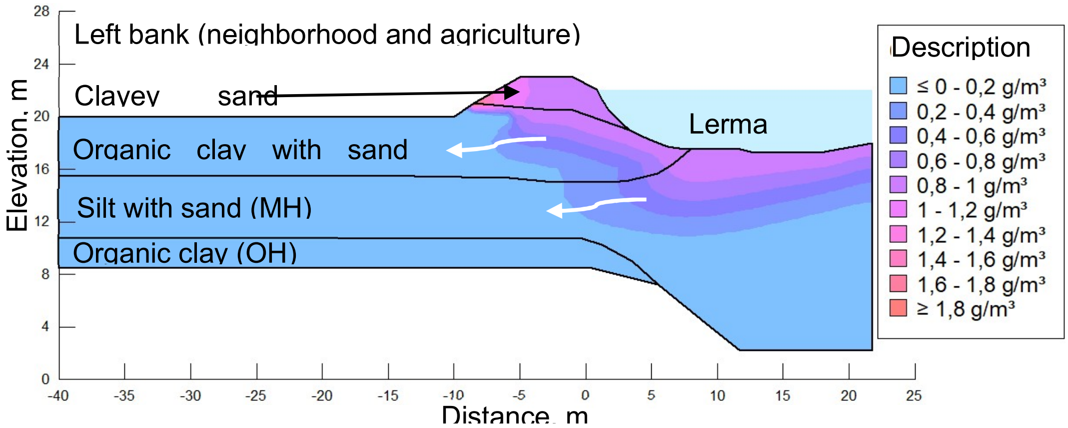


Figure 3.64 Plume of contamination for a 5-year period, left bank.

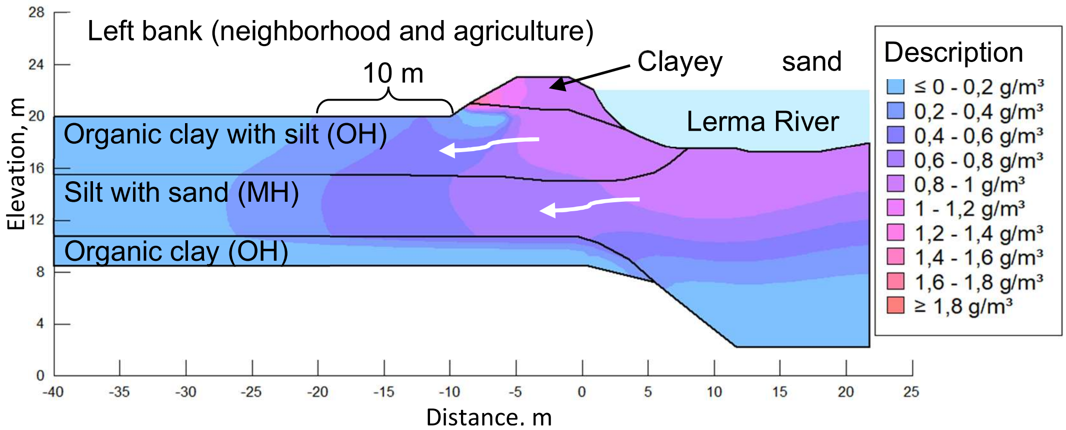


Figure 3.65 Plume of contamination for a 20-year time frame, left bank.

Figure 3.66 shows the graph with the results of the soluble lead concentration, for a time of 50 years, in the same figure the maximum permissible limits indicated in the Mexican standards/laws are specified by means of horizontal lines.

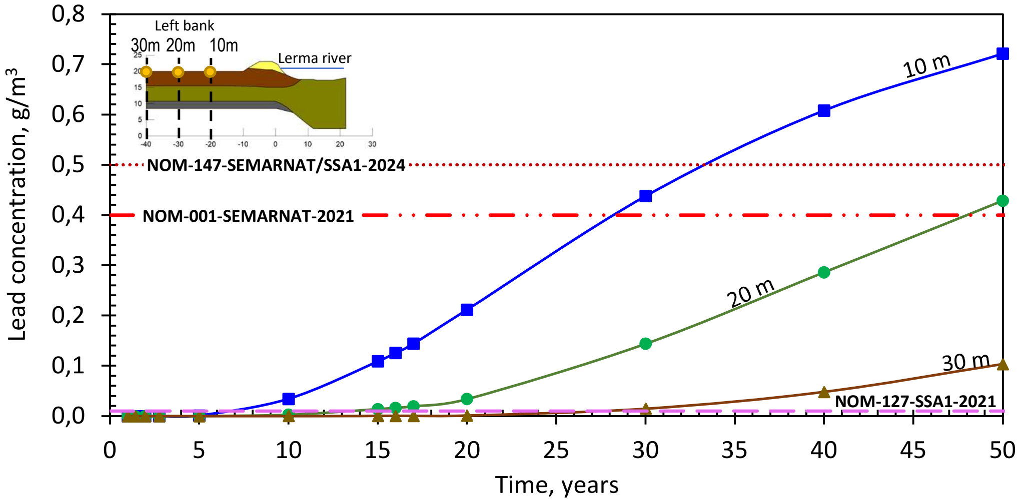


Figure 3.66 Graph of Pb concentration (on the surface of the left bank) versus time and its relation to Mexican standards/laws

The results revealed that:

1. Total soluble Pb concentration in subsurface water along the left side of the river exceeds the permissible limit of 0.01 gm^{-3}, making it unsuitable for consumption or irrigation.
2. Projection indicates that the pollution will take 28 years to extend 10 meters to the plain, exceeding the limits of Mexican laws (SEMARNAT, 2022; SEMARNAT/SSA1, 2007).

On the other hand, if the parameters listed in

Table 3.7 are considered and the same analysis criteria are assumed, except for the values in Table 3.9, to be replaced by a single dispersion ratio of α_L/α_T = 10/5 = 2 in all strata, then the results vary as shown in Figure 3.67. This figure shows that, when the same dispersion ratio (α_L/α_T =2) is assumed, with unique values of the longitudinal α_L and transverse α_T dispersion coefficients (α_L/α_T =10/5=2) in all strata, the pollutant expands in the subsurface at a faster rate.

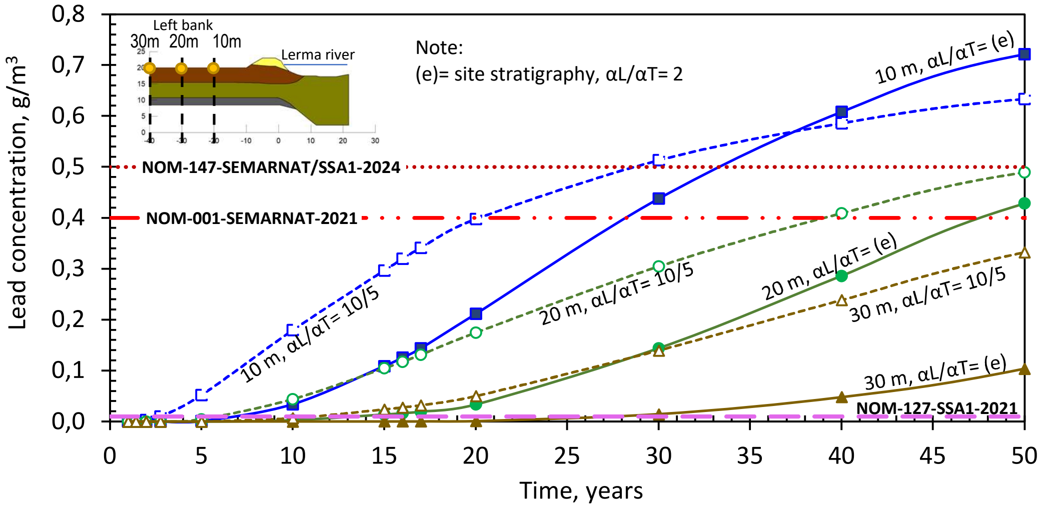


Figure 3.67 Graphic of Pb concentration (on the surface of the left bank) as a function of time, its relation to Mexican standards and its behavior with respect to the dispersion relation.

To evaluate the influence of permeability, values lower than those indicated in

Table 3.7 are assumed. These values are presented in Table 3.10.

Table 3.10 Geotechnical parameters for water flow analysis

Strata	SUCS	Description	Saturated permeability, K_{sat} (m/s)	Minimum water content, w_{min} (%)	Specific gravity, G_s	Void ratio, $e = wG_s$	Volumetric water content, $\theta_w = (e/(1+e))S$
E1	SC	Clayey sand	1.00×10^{-5}	24.60	2.64	0.65	0.39
E2	OH	Organic clay with sand	1.00×10^{-8}	148.20	2.13	3.16	0.76
E3	MH	Silt with sand	1.00×10^{-7}	143.50	2.66	3.82	0.79
E4	OH	Organic clay	1.00×10^{-9}	329.00	2.13	7.01	0.88

Note: *S* = Degree of saturation = 1 (saturated medium), SUCS = Unified Soil Classification System.

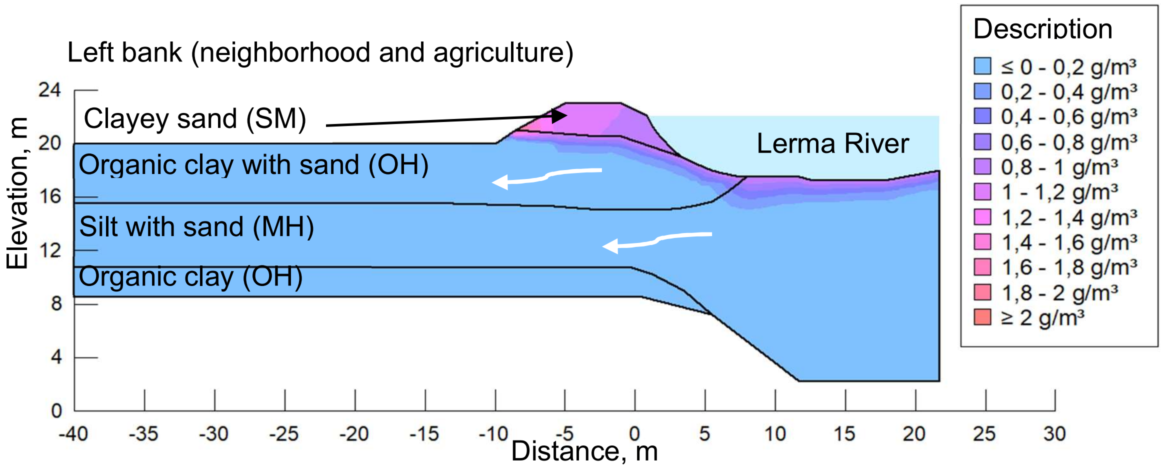


Figure 3.68 Plume of contamination for a 5-year period, left margin (low permeability in the strata)

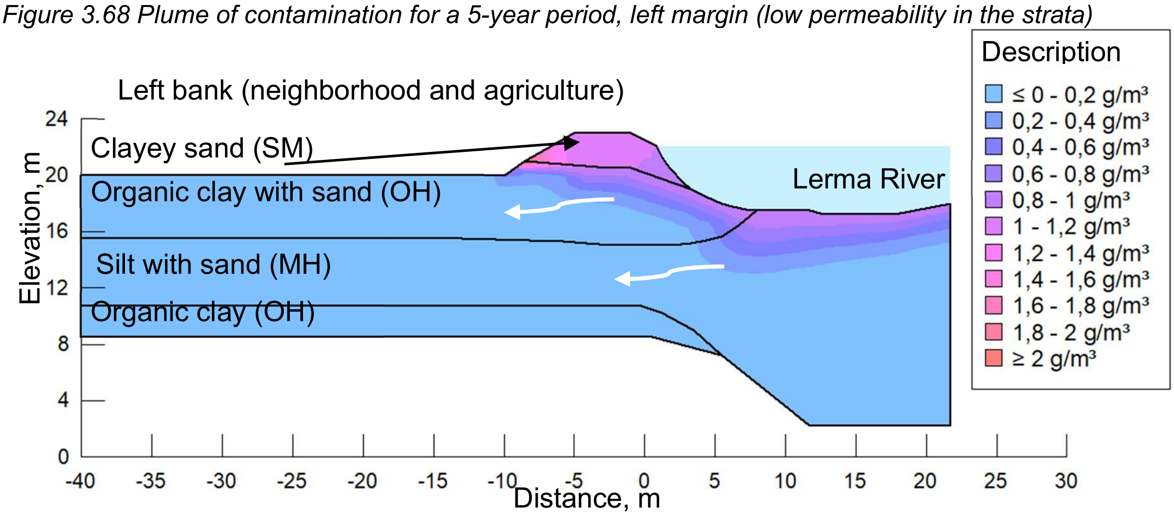


Figure 3.69 Plume of contamination for 20 years, left bank (low permeability in the strata).

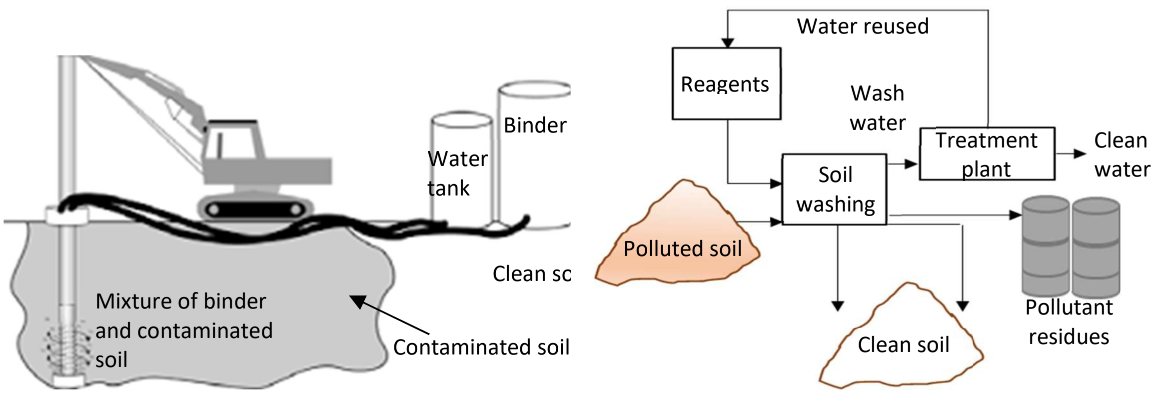
Subsequently, numerical modeling is performed, and the plume of contamination shown in Figure 3.68 and Figure 3.69 are obtained, for periods of 5 and 20 years, respectively. These figures show a significant difference with respect to the pollutant spots in Figure 3.64 and Figure 3.65, which makes it evident that permeability is a parameter with a significant influence in this type of analysis. In this case, in 20 years the pollution still does not spread to the left margin under study (because the strata have low permeabilities).

Remediation measure(s)

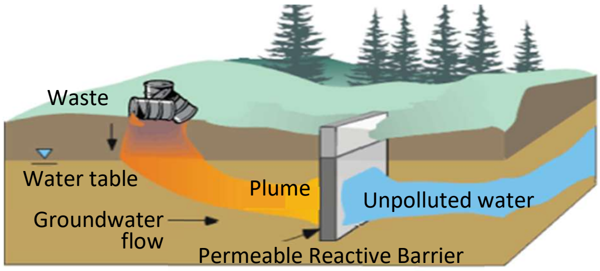
Soil remediation refers to operations performed to alter the composition of a pollutant by chemical, physical or biological actions in order to reduce the toxicity, mobility, or volume of polluted material. Remediation measures can destroy or modify pollutants, remove, or separate them from the polluted medium, and isolate or immobilize them to prevent their mobility (Iturbe-Argüelles, 2014). These techniques can be applied without excavating the soil (in situ) or in an excavated soil (ex situ). The most common techniques are listed below according to their approach.

Chemical techniques

- ***Solidification or stabilization***: Solidification encapsulates pollutants by adding binding agents to the soil, while stabilization makes the pollutants less toxic and immobile. It can be on-site, using machinery that injects the stabilizing agent to the depth of interest (Figure 3.70a); or the technique can be carried out off-site, by excavating and treating the polluted soil at another site, and finally returning it to the original site (Madhav *et al.*, 2024). This method is useful for treating heavy metals and improves soil strength, although it does not destroy the pollutants and is expensive.
- ***Soil washing***: In this technique, polluted soil is excavated and washed in a preset area with a specific solution, repeatedly, until the soil reduces its pollution to desirable parameters (Figure 3.70b). The USEPA states that it is not recommended to use this technique in mostly clayey soils; however, researchers at UNAM have proven that the technique works for soils with fines contents greater than 20% (Iturbe-Argüelles, 2014; Madhav *et al.*, 2024). It is effective in completely removing pollutants.
- ***Permeable reactive barrier***: It consists of constructing a wall by excavating a long and narrow trench, perpendicular to the path of groundwater flow (Figure 3.70c). This trench is filled with a reactive material to remove heavy metals (Environmental Protection Agency, 2021). It is economical and does not require surface treatment or water pumping. Over time, the barriers can become saturated with pollutants, requiring replacement of the reactive material.



a) b)

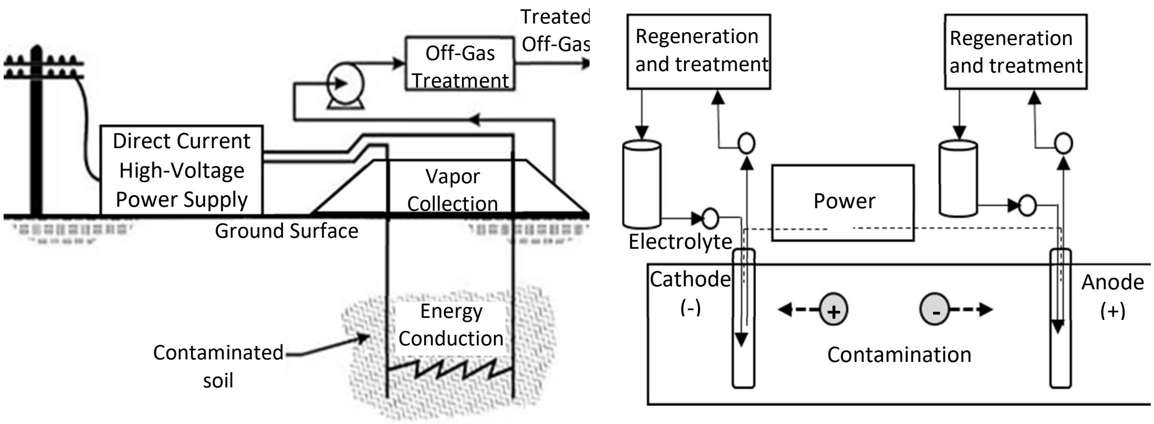


c)

Figure 3.70 Chemical soil remediation techniques a) solidification or stabilization (Environmental protection agency, 2003); b) soil washing (Martínez-Arreguín et al., no date) and c) permeable reactive barriers (Environmental Protection Agency, 2021)

Physical techniques

- ***Vitrification:*** Electrodes are used to raise the soil temperature between 1600 and 2000 °C (Figure 3.71a), which causes the decomposition or volatization of pollutants (Iturbe-Argüelles, 2014; Madhav *et al.*, 2024). It is effective for removing any type of heavy metals, but it is costly due to the high energy consumption it requires, and it can also cause subsidence due to the loss of soil volume.
- ***Electrokinetic:*** It is mainly used in soils with low permeability. It consists of applying a low intensity direct current between a positive and a negative electrode; pollutants travel through the water flow from the anode (+) to the cathode (-), as shown in Figure 3.71b, and the water is collected for treatment (De la Rosa, Teutli and Ramírez, 2007). Electrokinetics removes metals from the soil and works best when the pollution occurs by a single metal. It is very efficient in the laboratory (up to 100% effectiveness), but in the field it reaches 30%. It is expensive due to the consumption of electrical energy.



a) b)

Figure 3.71 Physical soil remediation techniques: a) vitrification (Fernández, 2023) and b) electrokinetics (De la Rosa, Teutli and Ramírez, 2007)

Biological techniques

The main biological technique is **phytoremediation**, which refers to the use of plants for soil remediation. Depending on the plant or the type of pollutant, the following methods can be used (Figure 3.72): Phytoextraction is when plant roots extract heavy metals and transfer them to the other parts of the plant. As the plants grow, they are harvested and taken to disposal sites, repeating this process until the desired heavy metal concentration is achieved at the site

(Santos-Ubaldo *et al.*, 2023; Madhav *et al.*, 2024). All remediation techniques are environmentally friendly and economical, but their effectiveness depends on the plant's adsorption capacity. Another process is *phytovolatilization* in which the roots absorb contaminants, transform them into volatile compounds and release them into the environment. *Rhizofiltration* (also called phytostimulation or plant-assisted bioremediation) is the decomposition of contaminants in the soil by microbial activity in the rhizosphere. Finally, *phytostabilization* occurs when the plant only immobilizes contaminants by absorption or accumulation in the root.

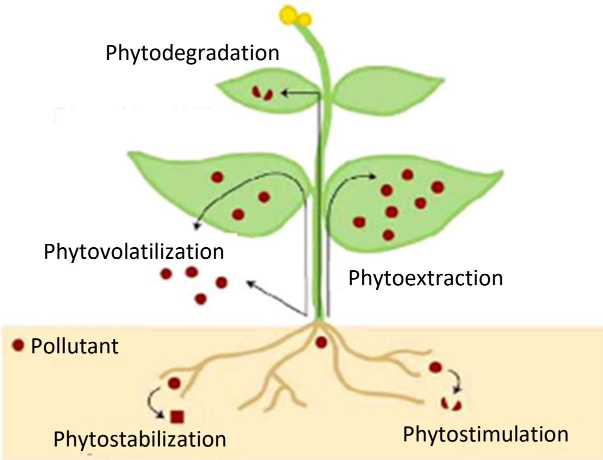
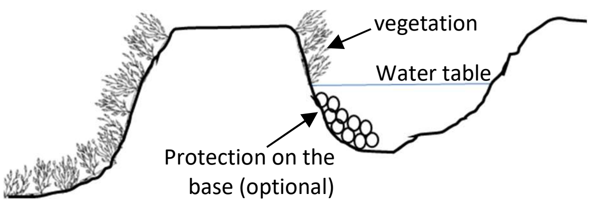


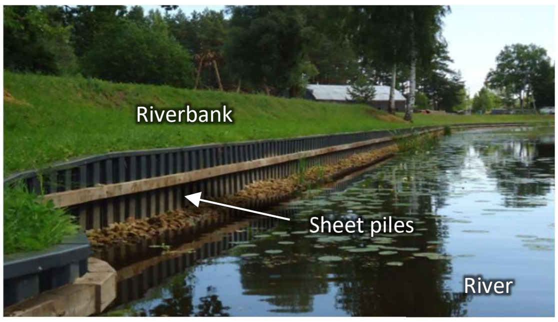
Figure 3.72 Processes that can occur in the soil bioremediation technique.

Engineering techniques

- **Marginal slope protection with vegetation:** consists of placing plants on the surface of the embankment slope to provide stability, Figure 3.73a (CONAGUA, 2000). It is a low-cost technique and, depending on the plant, it can contribute to pollution mitigation by using any of the methods indicated in the biological techniques section.
- **Sheet piles:** These are pieces of wood, steel or reinforced concrete that form a retaining wall to stabilize edges (Figure 3.73b) and slow the flow of pollution. They do not remove pollutants by themselves, but can be combined with other techniques previously mentioned to mitigate pollution.



a)



b)

Figure 3.73 Engineering techniques to mitigate soil contamination a) marginal protection with vegetation (CONAGUA, 2000) and b) sheet piles (Jasinski, 2024)

An adequate protection and remediation design must consider some key points. First, the extension of the plume of contamination defines the polluted area. This delineation should limit land use and activities to safeguard health and ecosystems as long as contamination exists, and represents the minimum width for remediation.

Second, designing and installing a *permeable reactive barrier* is recommended to filter contaminated groundwater flow toward adjacent plain areas (settlements and peri-urban farmland). Silica sand from the coasts of Alvarado, Veracruz (Mexico), has proven effectiveness in removing heavy metal contamination (Susunaga Miranda *et al*., 2021) and could be used for this purpose.

Third, a mixed technique for remediation is convenient. Implementing a riparian zone with native trees for phytoextraction is recommended. For instance, *salix bomplandiana*, which is a threatened Species under monitoring for the International Union for Conservation of Nature (IUCN Red List, 2018), has shown potential for phytoremediation (Mohsin *et al*., 2022; Pulford & Watson, 2003). Additionally, using *vetiver grass* for slope stabilization and phytoremediation is advised due to its high capacity to absorb and accumulate heavy metals, including lead, in its roots (Truong & Danh, 2015); vegetated area should be fenced to prevent cattle (see Figure 3.60) from grazing on the grass. An example of the use of this technique occurs in the Oaxacan Mixtec region of Mexico, where there are high levels of deforestation, soil deterioration and water scarcity, then, to capture rainwater and promote soil restoration, a series of strategies have been implemented, among which the use of living barriers with vetiver grass stands out. Similarly, in this region, this technique is used to stabilize roads and prevent erosion problems on slopes (Centéotl, no date).

The proposed protection and remediation techniques are general recommendations. Both can significantly contribute to the restoration of contaminated areas. Site-specific measurements and analysis are necessary to ensure a custom design. Additional techniques, such as soil washing, may be considered depending on the timeframe for remediation and goals. Monitoring heavy metal concentrations during the remediation process is relevant to evaluate and report the effectiveness of the implemented technologies.

Other information about this case

There is a delineation criterion of 10 meters in Mexico for flooding protection called "federal zone" (Ley de Aguas Nacionales, 1992). However, this horizontal strip could be short of prohibiting human activities where water pollution and differences in level between water elevation and adjacent plain areas create a hydraulic gradient toward the lowland.
The study's relevance lies in its being the first of its kind. Considering the date of the first report published by Avila-Perez & Zarazua-Ortega (1993) and CONAGUA (1996), there are reasons to assume that the subsurface zone has been polluted for about 30 years. Based on this, future studies could focus on compare and the data obtained from the numerical model with data collected *in situ*.

The proximity between the river and crops represents a potential health and environmental risk due to farmland activities (Figure 3.74) in contaminated areas due to toxicity, even in low concentrations (Rain *et al*., 2022). Due to this, knowing the extent of pollution will allow the delineation of a strip to guarantee that land management is safe.

Numerical modelling of pollutant transport helps to understand how concentrations vary over time and at different distances from the river in the region studied. This understanding makes it possible to determine the spread of pollution and to propose land use limits to protect the well-being of inhabitants and land users. Based on the results obtained in the numerical modelling, it was observed that the most important variables influencing this type of analyses are permeability and dispersion coefficients. Different soil remediation measures were described to help prevent, mitigate or eliminate soil contamination.

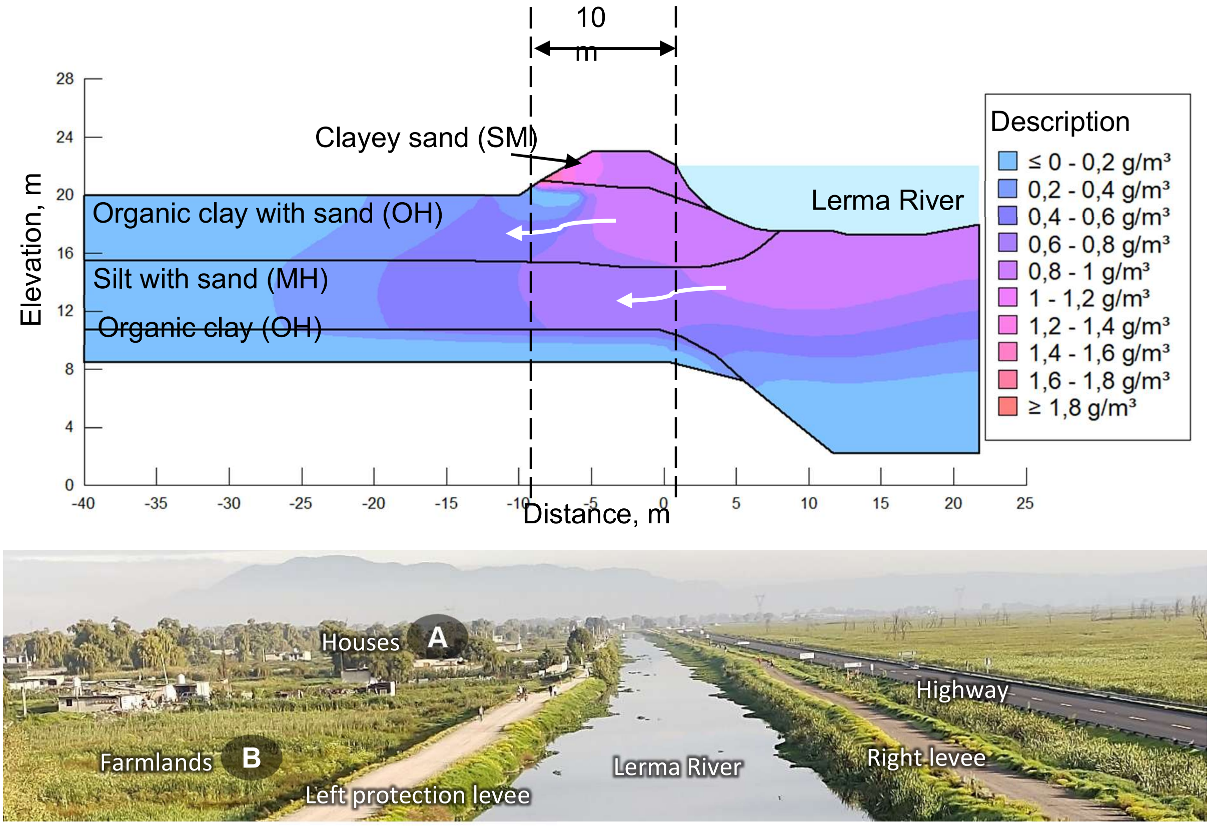


Figure 3.74 Top: Pollution dispersion in soils (20 years) on plain areas adjacent to the Lerma River in San Mateo Atenco (Mexico). Below: Image indicates land use. A and B are houses and farmlands, respectively.

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3.1 9 Case description: Geosynthetic honeycomb cells filled with concrete for spillway at the Intabo levee (external erosion)

Authors: José A. Walters Monteiro, TECNICA Engenheiros Consultores, Lda, Mozambique.
Keywords: Failure mechanisms: External erosion on the water side; River levees; Subsurface: sand, clay; Climate change, Population growth.

For the corresponding technique factsheet, please refer to Section 3.2.6.

Setting

The Lower Licungo region in Nante, Zambézia Province (Mozambique), is highly vulnerable to flooding due to its geographic location near the river mouth and the flat relief that limits rapid outflow to the sea. The Licungo River basin covers approximately 27,000 km², with headwaters in the Gurúè highlands and the Tacuane mountains, both areas of intense rainfall (about 2,000 mm/year). In January 2015, an exceptional flood occurred, caused by intense local rainfall of around 600 mm in just ten days, combined with upstream contributions from the Licungo and its tributaries.

At Mocuba (hydrometric station E91), which provides the most reliable records upstream of Nante, the flood peak in 2015 exceeded the maximum gauge limit of 12.0 m. Based on post-event assessments, the corresponding discharge is estimated at about 19,000 m³/s, which, according to flood frequency analysis, corresponds to a return period in the order of decades to centuries, thus confirming its extreme rarity. The flood inundated the entire lower Licungo plain, overtopping and breaching earth embankments (levees) in several sections, including a 170-m breach downstream of the Intabo pumping station.

Historically, protective dikes in Nante were constructed to safeguard irrigation systems, typically designed for return periods of 10–20 years. However, demographic growth and the destructive floods of 2015 have highlighted the need to reassess the degree of flood protection provided to the population and to consider more resilient solutions. This case study refers to the resilient solution designed to protect the Intabo pumping station.

Historical background of Intabo Pumping Station and levee development

The Intabo pumping station was constructed in the Portuguese colonial time to support the Intabo irrigation scheme on the left bank floodplain of the Licungo River. Prior to its construction, the site was naturally protected by a riverbank. During construction, the natural bench was reshaped into an engineered levee to provide formal flood protection.

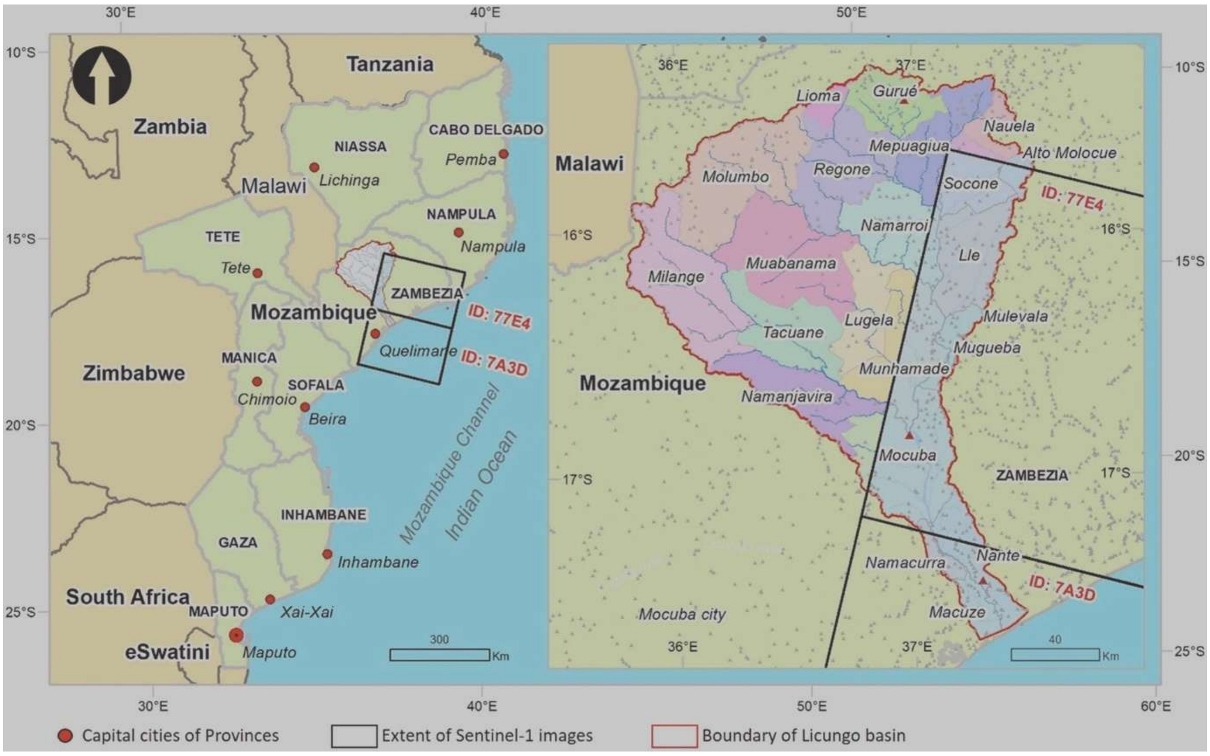

Figure 3.75 Location of Nante within the Licungo River basin. Source: Journal of Hydrology

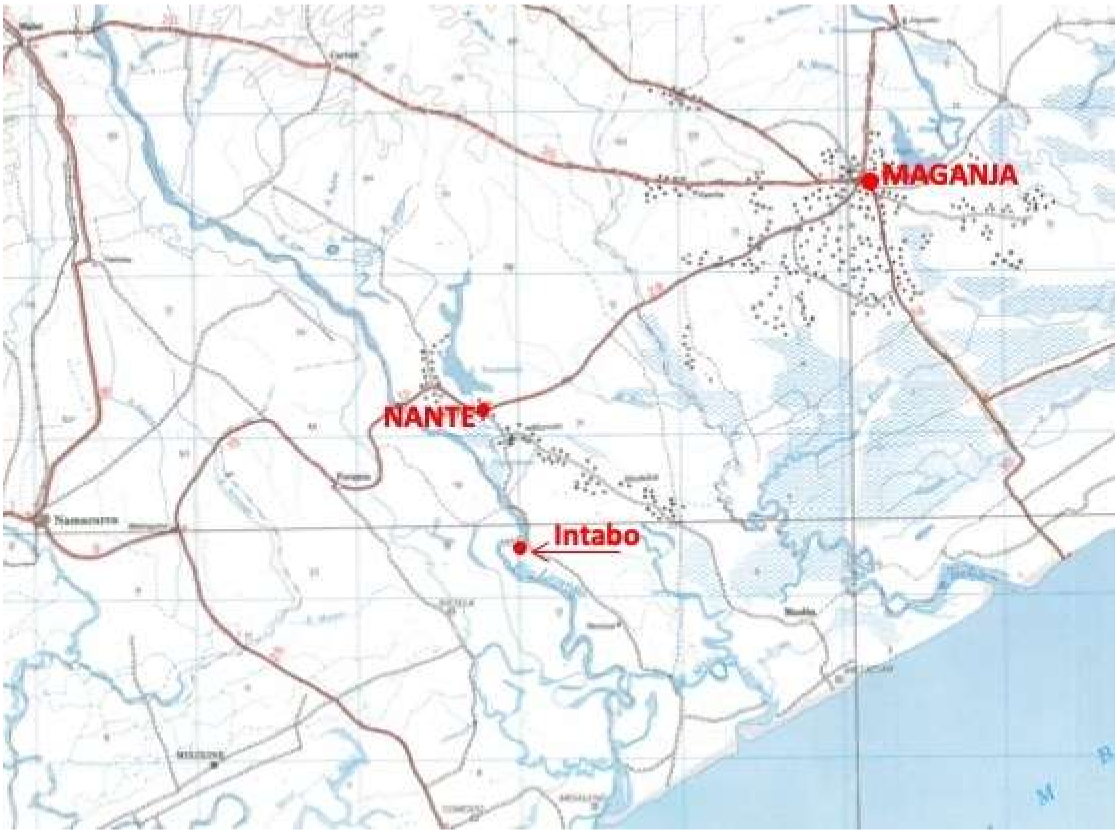

Figure 3.76 Topographic map of Maganja da Costa District and the location of Intabo. Source: CENACARTA

Figure 3.77 Satellite imagery of the 2015 flood in low Licungo region, Nante

Problem description

The 2013-2014 floods in the low Licungo region of Nante caused some damages to the protection levees as well as to the Licungo East riverbank. The stretch of the riverbank downstream the Intabo pump station was seriously eroded on the river side. In January 2015, the extreme flood of the Licungo River caused a breach of approximately 170 m length in this levee section, leaving the portion adjacent to the pumping station in unstable condition. Failure mechanisms included external erosion on the water side and breach initiation at locally weak sections.

In response, as part of an emergency recovering program for the levees of Nante region founded by the World Bank, protection works were designed and implemented downstream of the station, consisting of a surface spillway made of geosynthetic honeycomb cells filled with concrete. Additional measures included toe and lateral slope protection with gabions, as well as the construction of a stepped cascade and a gabion-lined discharge channel adjacent to the structure.

Figure 3.78 The stretch downstream of Intabo pumping station in September 2013 and November 2014

Figure 3.79 The aftermaths of 2015 floods in Intabo, showing the major breach.

Figure 3.80 The condition of the remaining riverbank downstream de Intabo pumbing station

Figure 3.81 Erosion in the land side of the pumping station

Remediation measure(s)

Description the selected remediation methods

The primary remediation adopted at the Intabo levee in 2016 was the construction of a surface spillway formed by geosynthetic honeycomb cells filled with concrete, anchored into a buried gabion toe to mitigate scour, and complemented with gabion sidewalls. The design also included a stepped cascade made of concrete-lined gabions and mattresses, and a gabion-lined discharge channel downstream of the land side to limit erosion during drawdown or overtopping splash.

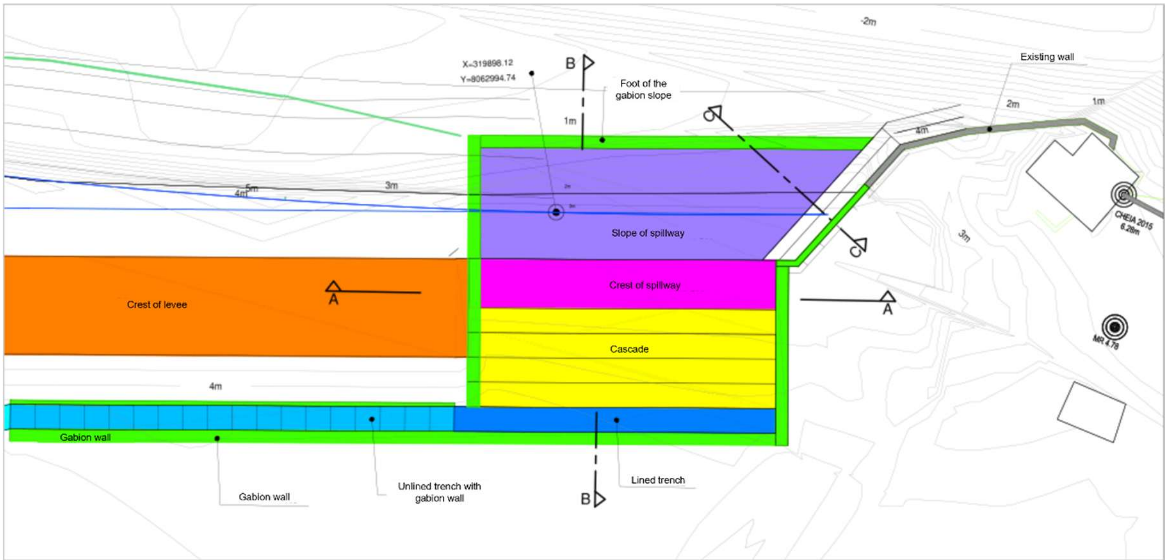

Figure 3.82 Plan view of the Intabo Spillway

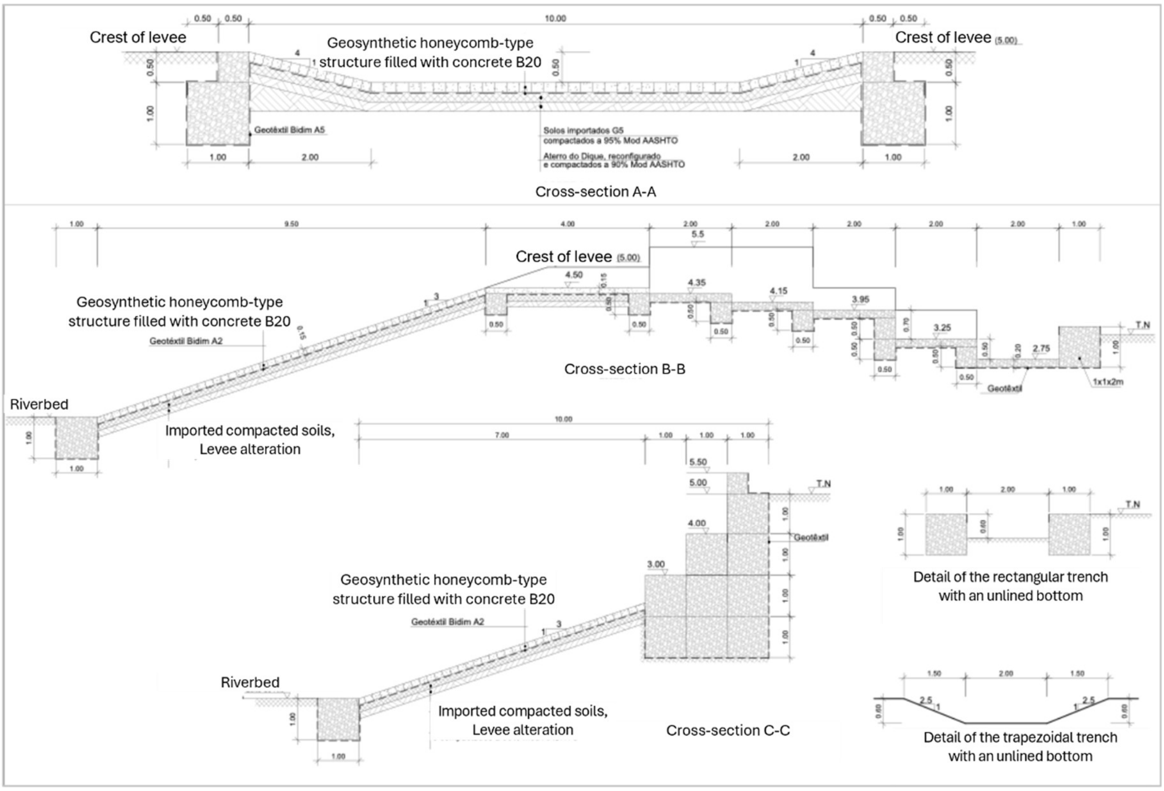

Figure 3.83 Cross sections of the Intabo Spillway

Performance
The structure has been tested by subsequent floods in 2019, 2020 and 2022 and remains intact. Field inspections and drone imagery confirm its effectiveness.

Figure 3.84 Birds eye view of the Intabo Spillway after the 2022 floods

Other information about this case

For the adjoining downstream reach of the levee, a more economical alternative was proposed, consisting of gabion toe protection combined with a vegetated slope cover to reduce superficial erosion. However, this complementary measure was not implemented due to budget constraints, leaving the concrete-armoured spillway as the main reinforced section of the levee.

Maintenance considerations include:

- periodic inspection after high-flow seasons;
- debris clearance;
- joint and toe checks.

Extreme flood considerations include:

- Contingency planning for exceptional events (overtopping, prolonged saturation);
- Emergency stock of gabions, geotextiles and cement.

This technique should be replicable to similar alluvial levees along Mozambican rivers with local adaptations.

3.1.10 Case description: Concreted rip rap at the Rhône Delta (external erosion)

Authors: Thibaut MALLET, SYMADREM, Arles, France.
Keywords: External erosion on the land side; River levees; stiff clay; increasing hydraulic loads (intensity or frequency) due to climate change.

For the corresponding technique factsheet, please refer to Section 3.2.9.

Setting

The Rhône delta is located in south-eastern France, downstream of the catchment area. It is protected by two hundred and twenty five kilometers (225 km) of river levees and fifty kilometers (50 km) of sea levees, managed by SYMADREM, a local authority. One hundred thousand (100,000) people permanently live in this area. This population triples during the tourist season. After two catastrophic floods occurred in the mid-nineteenth century (1840 & 1856), levees were substantially raised and designed as levees systems in place of other even older structures. By this time, the narrative of the unsinkability of the levees has taken root in the population. Levees had to be raised to an elevation so high they would never be overtopped by the Rhone river.

Seven breaches due to concentrated leak erosion caused by the floods of 1993, 1994 and 2002 caused three floods in quick succession, the first floods in 137 years. These floods highlighted the intrinsic weakness of the levees, which were compacted by hand without taking into account the optimum water content: a weakness exacerbated by the recurrent presence of badger burrows and numerous hydraulic structures crossing the levees.

But it was the flood of December 2003 (Figure 3.85) that revealed the need for a paradigm shift. This almost ‘centennial’ flood, caused four major breaches in the levees and the release of 227 million m³ of water into protected areas, flooding 12,000 inhabitants and causing €700 million in damage.

Figure 3.85 Flooding of the Rhône delta in December 2003 (© SDIS13 and J. Roche)

This flood, the worst since mid-nineteenth century, was the starting point for a collective awareness: the narrative of the unsinkability of levees was no longer appropriate to face the intensification of extreme phenomena. A paradigm shift was needed. The Rhône Plan, supported by the State and the local authorities has introduced a radical change: no longer aiming for illusory unsinkability, but building a resilient system that accepts flooding, anticipates it, organizes it and makes it controllable.

Rather than raising the levees, which was the response of the public authorities after each flooding, the following principles have been adopted:

- Accept flooding for rare floods (100-year floods upstream of Arles and 50-year floods downstream of Arles), with an equal distribution of the volumes discharged between banks.
- Consider breach formation up to the millennial flood as unacceptable,
- Ensure the rapid drainage of spilt water towards the sea, using drainage systems and complementary hydraulic infrastructures.

After 150 years of belief in the unsinkability of levees, the social acceptance of such a project was not a small matter. The paradigm shift from a high risk of flooding due to breaches, unpredictable in terms of occurrence and random in terms of location, towards a low but certain risk of controlled overflowing have required many concertation meeting and met sometimes virulent opposition.

We had to reinvent our communication. Semantics had to be adapted. The term of ‘levees resistant to overflow’ was preferred to the term ‘spillway’ or even ‘overflowing levee’, which were perceived as an aggression by rural populations, who felt sacrificed for the benefit of urban populations. Our public communication focused on the negative consequences of inaction and the benefits of action.

Thanks to hydraulic modelling (Figure 3.87), educational figures (Figure 3.86), in-depth studies of alternative solutions asked by some associations and the transparency of public choices, we have gradually won the support of the population and succeed to start the works without opposition.

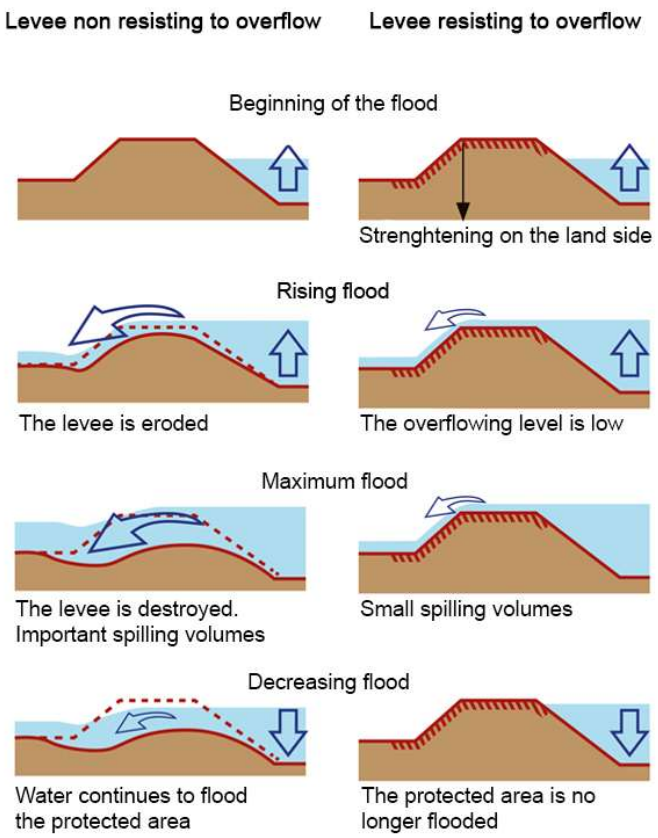

Figure 3.86 Comparison between levees system without and with levees resistant to overflow

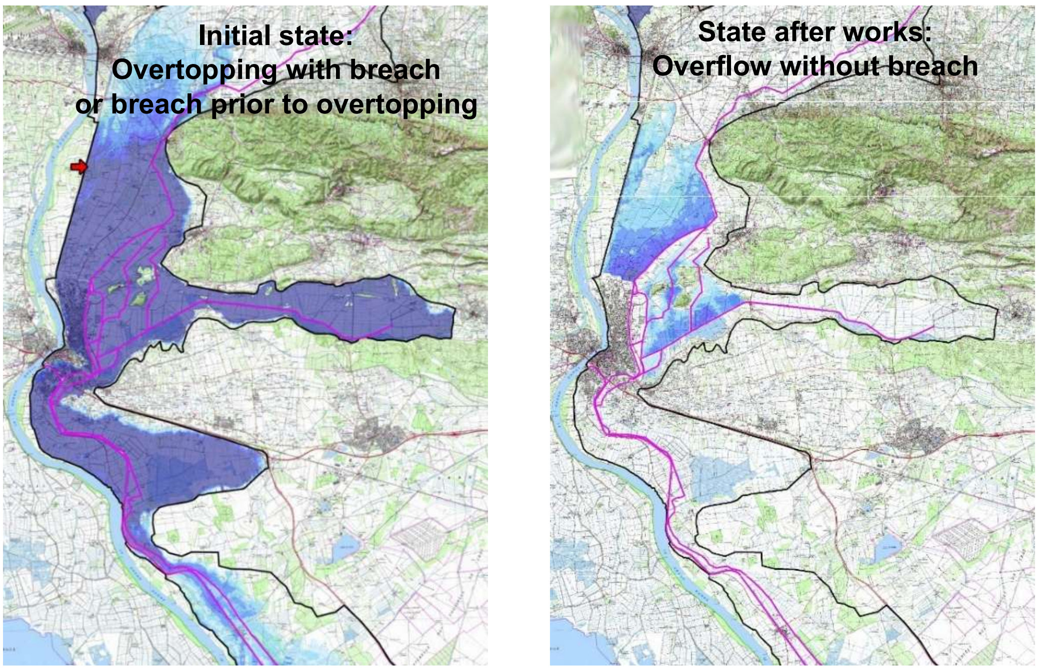

Figure 3.87 Inundation mapping with and without breaches for the left bank levees system

Between 2008 and 2023, 73 km of river levees have been secured, including 10 km of levees resistant to overflowing. This work represents an investment of €225 million, out of a total of €450 million planned.

Problem description

Historical feedback based on 8 inundations since 1840 revealed a probability of breach of 50% for floods with a return period of 20 years and a probability of breach of 100 % for flood with a return period of 40 years. These probabilities have been confirmed by probabilistic approach built by SYMADREM for hazard studies, name used for risk analysis required French regulation.

Risk analysis has confirmed two main failure modes in the Rhone delta levees: internal erosion (concentrated leak erosion) and overflowing. In 2018, in-situ overflow tests to compare overflowing resistance between untreated soils and soil treated with quicklime have confirmed the low resistance to overflow of Rhone delta levees, even compacted to current best practices (Figure 3.88). In those conditions, overflow resistance must be improved to increase safety and durability of levees.

Figure 3.88 Overflow tests carried out by INRAE in 2018 on a test levee compacted to current best practices

Remediation measure(s)

The new system is based on three hydraulic levels (Figure 3.89):

- The protection level: which corresponds to the level reached by floods with a return period of around 100 years upstream of Arles and 50 years downstream, which themselves correspond to the low points of the old levees systems.
- The safety level: which corresponds to the level reached by the millennial flood .
- The danger level: which corresponds to the safety level with a 50 cm freeboard, for which the system may suffer non-structural damage.

This involves securing the entire levee system up to the 1000-year flood, implementing spillways on levees and draining the volumes of water spills to the sea.

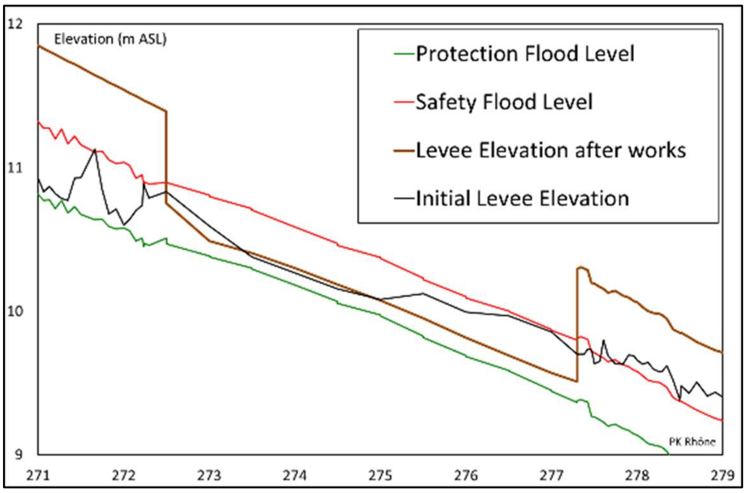

Figure 3.89 Principles of altimeter setting between old and new levees.

3.1.11 Case description: Revetment and berm using granular material along the Tisza River (slope stability)

Authors: Edina Koch, Richard Ray (Széchenyi István University), General Directorate of Water Management

Keywords: Slope sliding; river levees; soft clay; sandy silt; silty sand; increasing hydraulic loads (intensity or frequency) due to climate change.

For the corresponding technique factsheet on revetment and berm using granular material, please refer to Section 3.2.11.

Setting

Hungary has one of the extensive river dike systems in Europe along the two major rivers (Danube and Tisza) and their tributaries. There are 12 regional Water Directorates responsible for flood protection.

Covering an area of 157,186 km², the Tisza River Basin is the largest sub-basin of the Danube River Basin. The Tisza River is the longest tributary of the Danube (966 km) and has the second-largest flow after the Sava.

The Lower Tisza District Water Directorate is a state-owned water management institution responsible for operating and maintaining facilities in Hungary's Lower Tisza Valley drainage basin. The directorate's main activities include flood prevention and control, drainage and mitigation of inland excess water, provision of irrigation water, and the sustainable management of surface and groundwater resources (Figure 3.91).

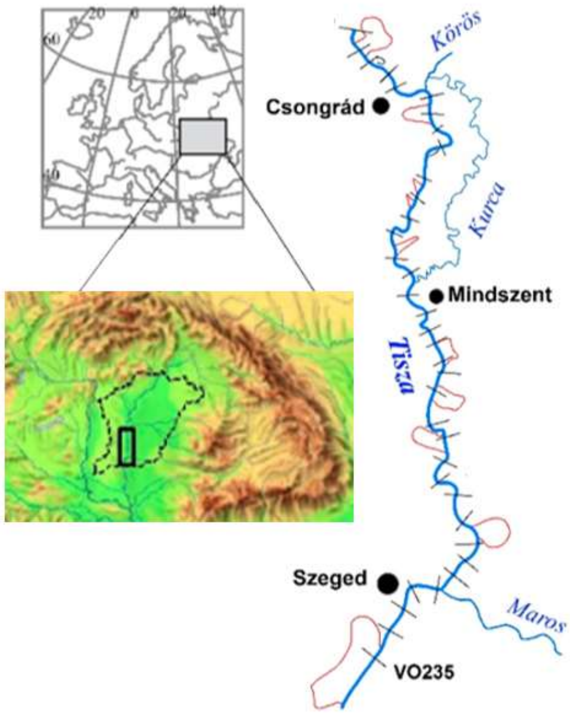

Figure 3.90. Location of the investigated case

The Lower-Tisza Region Water Directorate (ATIVIZIG) protects ~330 km long primary dyke:

- ~ 200 km along the Tisza River
- ~ 35 km along the Hármas-Kőrös
- ~ 95 km along the Maros River

ATIVIZIG has five river basins, three of which extend into neighbouring countries, resulting in shared embankments with Romania and Serbia.

The investigated case is a rural levee. The measured levee's current height is 6.5 m, and the base width is 45 m, with a crest 4.5 m wide. The landside slope of the levee is 1:3, and at 2/3 of the height, there is a 3.5 m wide berm (Figure 3.91) .

The top layer of the levee consists of low-plasticity clay, the foundation and the embankment body are composed of high-plasticity clay. The high-plasticity clay is underlain by layers of low-to-medium plasticity clay, followed by silty sand and sandy silt at greater depths.

A typical high-water event lasts for about 3-4 weeks.

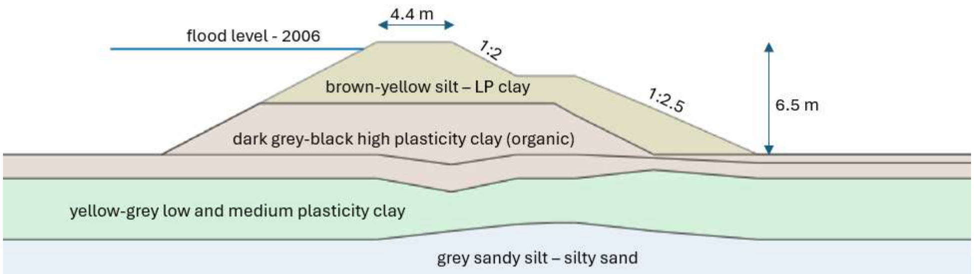

Figure 3.91. Geotechnical model of the investigated case

Problem description

The case is a slope failure induced by the April 2006 flood near the Tisza River. The failure occurred on the landside of the levee (Figure 3.92) and required immediate stabilization by a sandbag revetment at its toe.

Before the flood, the levee was in good condition. Observations of field performance pointed to the apparent failure mechanism where seepage helped to generate reduced effective stress and strength. The slope (berm) moved just as the water level peaked, rising 1.5 m in about a week. The water seeped over the lower, less permeable (high-plasticity) black clay material and through the upper, more permeable (low plasticity) yellow clay zone. This process weakened the strength of the black/yellow clay interface and, to some extent, the yellow layer. At the same time, seepage gradient forces almost parallel to the downslope direction increased the sliding forces.

Geotechnical finite element modelling was applied to understand the failure mechanism.

Figure 3.92 Slope sliding and temporary reinforcement

Remediation measure(s)

Description of the selected remediation methods

After the flood, repair crews reinforced the failed slope using a revetment on the land side for long-term stabilization. The coarse-grained material and a drainage system provided seepage control along the landside levee toe.

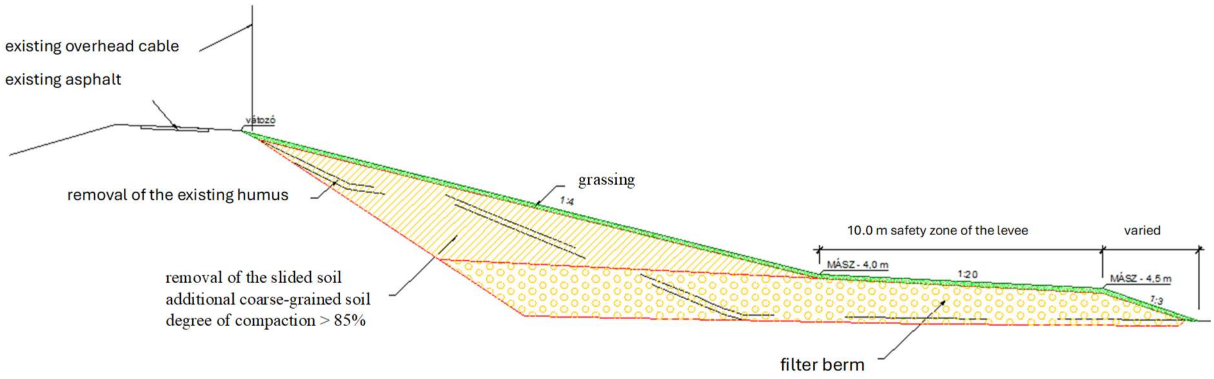

Figure 3.93 Cross section of the reinforcement of the case

It is a common practice if the landside slope is steeper than 1:3.

Performance

The reinforcement of the levee occurred after the 2006 flood. Since then, several floods (with similar levels) have inundated the region, but there have been no problems with the reinforced levee. A revetment using granular material has provided an effective method for levee stabilization.

Other information about this case

Due to the extreme flood event in 2013, engineers have raised the design flood parameters. The event has prompted new values for design water levels, evaluations of new levee designs, and reinforcement of existing levees.

3.1.12 *Case description: Increase of subsoil strength with vacuum consolidation at the Markermeer levee (slope stability)*

Corresponding author: J.W. Dijkstra, Cofra, The Netherlands (jeroen.dijkstra@cofra.com)
Keywords: Vacuum consolidation

For the corresponding technique factsheet on increase of subsoil strength with vacuum consolidation, please refer to Section 3.2.13.

Setting

In the Netherlands, a substantial dike improvement program, known as HWBP (High Water Protection Program), with a budget of several billion euros, is underway and is planned to continue until 2050. This program is experiencing a growing demand for innovative solutions aimed at enhancing the macro stability of dikes in a manner that is both time and cost-efficient. The stimulus for this improvement program stems from several factors, including the rise in water levels due to climate change, the enforcement of stricter safety regulations, and the introduction of new calculation methods. The need for innovative solutions is driven by the aim to reduce costs, shorten project timelines, accommodate limited working spaces, preserve historically significant landscapes, and manage an increasing number of project interfaces.

As part of the HWBP, significant research initiatives have been established, including the POVM (Project on Macro Stability). The POVM specifically focuses on slope stability and has initiated various research projects. One notable initiative involves exploring the use of vacuum consolidation to bolster the strength at the base of the embankment, with the goal of minimizing the impact of dike strengthening projects on the landscape and reducing the need for materials and transportation.

While vacuum consolidation is a well-established technique in infrastructure projects for reducing residual settlements and horizontal deformations post-construction, its application specifically to increase the strength at the base of the dike is less common. The resulting increase in strength can render berms of the dike obsolete or allow them to remain at lower levels, thereby limiting the use of embankment materials and reducing the impact on historically significant landscapes. This aspect was particularly crucial in the Markermeer levee reinforcement project, where the preservation of the famous and historically significant peat landscapes and dike villages just north of Amsterdam, the Netherlands, was of paramount importance.

Problem description

The potential benefits of the system prompted the Markermeerdijken project, a collaboration between Waterboard Noord Hollands Noorderkwartier and contractor Boskalis, to undertake a vacuum consolidation trial in partnership with the research program POVM and ground improvement contractor 3.2.13.

The specific objectives of the trial for the POVM were to establish guidelines for the technique concerning calculation methods, verify whether the increase in pre-consolidation stress and shear strength, following the removal of vacuum pressure alone, is permanent, and to investigate the applicability of the SHANSEP method, as proposed by Lad & Foot (1974), to calculate the strength enhancement achieved through vacuum consolidation.

Remediation measure(s)

Description of the selected vacuum methods

The trial involved the selection of two vacuum consolidation techniques: the Beaudrain-S and the traditional liner vacuum consolidation system.

Figure 3.94 provides an illustration of both techniques along with the local soil profile of the site.

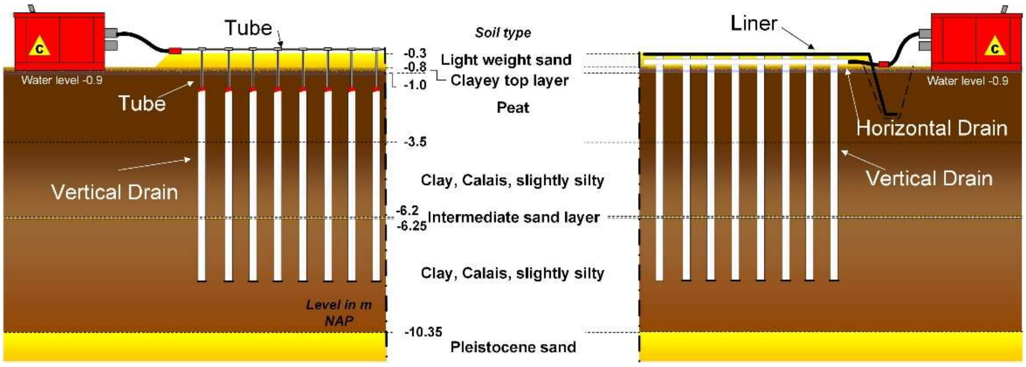

Figure 3.94 Cross section of techniques. Left: Beaudrain-S, Right: traditional vacuum system

The Beaudrain-S system comprises individual drains that are formed from a combination of a prefabricated vertical drain (PVD) section and a tube section. The length of the PVD is specifically designed for application in the compressible layers below the groundwater, while the remaining soil layers, including those above the water table and those posing a risk for air or significant water inflow, are addressed using the tube. On the surface, the individual tubes of the combined drain-tube system are connected using T-couplings and tubes, which are then linked to a vacuum pump situated at the field's perimeter.

In contrast, the traditional liner vacuum system utilizes conventional PVD, horizontal drains, and a membrane/liner. The vertical drains are installed from a permeable working platform and connected at the surface to horizontal drains, which in turn are connected to a closed collection drain. This collection drain is, in turn, connected to a vacuum pump. The entire area is sealed by an impervious membrane/liner, which is buried along the edges of the area using peripheral trenches below the water table.

Post consolidation testing

The consolidation testing was conducted over a period of 7 days and following 100 days subsequent to the shutdown of the pumps and the cessation of vacuum pressure. Throughout this investigative phase, ball penetrometer tests and additional boreholes were carried out to study the altered post-consolidation stress, the resulting OCR value, the SHANSEP parameter m, and the adjusted shear strength.

Ball penetrometer results

Figure 3.95 shows a typical ball penetrometer results before start of the trials (t=0), 7d (t=7) and 100d (t=100) after the vacuum consolidation has been applied and removed on the Beaudrain-S section.

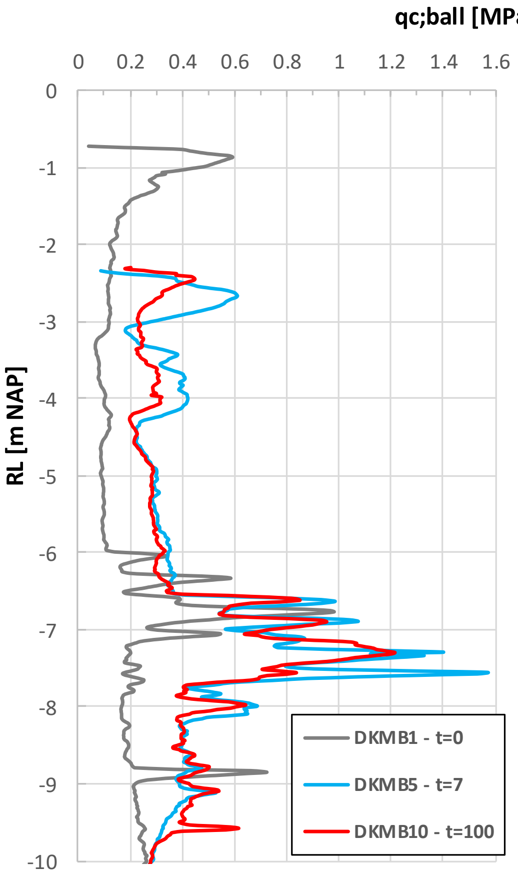

Figure 3.95 Ball penetrometer results before and after consolidation in the Beaudrain-S section

The ball cone penetration resistance has clearly increased by approximately 200-250% following the vacuum consolidation period. While there is a slight decrease in penetration resistance over time in the top peat layer, the cone resistance remains significantly higher than the initial value. It's worth noting that this variation could also be influenced by local variability in the subsoil, as a post-test was conducted at a slightly different location. Overall, the cone penetration resistance is directly linked to a notable increase in soil strength.

Pre-consolidation stress and undrained strength

The increase in pre-consolidation stress was directly determined through compression tests on soil samples, collected from boreholes before and within 7 days after the removal of vacuum. It's important to note that the proximity of the boreholes to the vertical drains impacts the pre-consolidation stress. As a result, the boreholes were strategically positioned in the theoretical center of the drainage grid, representing the most conservative location in terms of strength increase. However, due to settlement, the exact positioning of the drains in relation to the boreholes remains unknown.

In addition to laboratory testing, the pre-consolidation pressure was indirectly determined by correlating it with the ball cone resistance. By analysing the achieved water pressure reduction, the initial stress level, and pre-consolidation pressure, the increase in pre-consolidation pressure was estimated using established calculation methods. The results of the calculation indicated that the shear strength of the peat layer in the traditional vacuum area increased from

11 kPa to approximately 25 kPa, and in the Beaudrain-S area, from 11 kPa to about 40 kPa. For the silty clay layers, there was an increase from 20 kPa to 70 kPa in both areas.

The calculated increase in pre-consolidation stress in both the peat and clay layers closely aligns with the increase observed in laboratory tests (direct measurement based on K0-CRS tests) and field tests (indirect measurements based on ball-penetration tests), as shown in Figure 3.96. The pre-consolidation stress is derived from the ball-penetration test using correlations (n_{ball}) between undrained shear strength (derived from DSS tests) and ball resistance. Within the SHANSEP method, the pre-consolidation stress can be calculated based on the S-ratio of undrained shear and the actual effective stress..

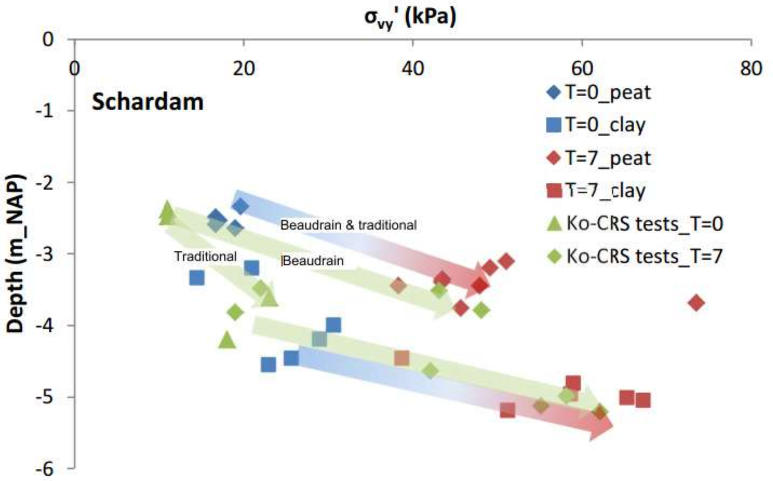

Figure 3.96 Direct (K0-crs test, green line) & indirect pre-consolidation stress determination (Ball-penetration test, red and blue lines) for different soil layers at t=0 and t=7 (within 7d after ending vacuum), source: POV-M (2017)

Triaxial and DSS tests have been performed to determine the strength increase. These results have been compared to the SHANSEP method and to the correlation with Ball cone resistance. Results were found to be comparable, please refer to Figure 3.97 for an example.

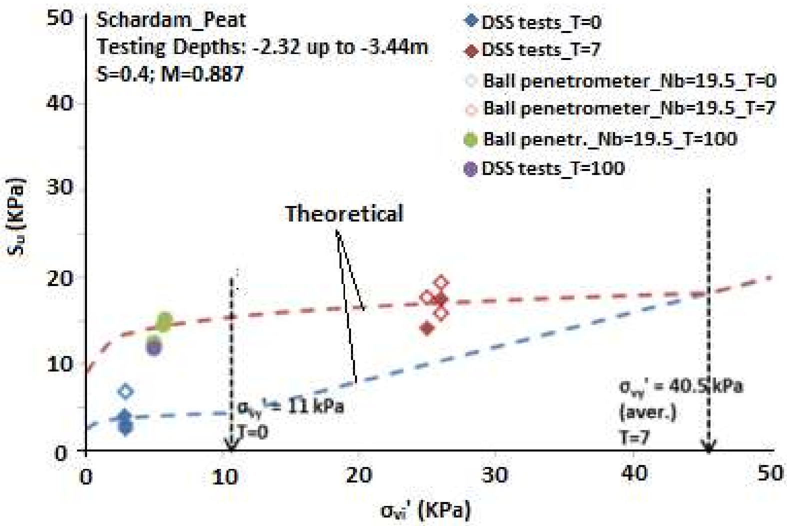

Figure 3.97 Example of Beaudrain-S area (Peat): DSS results, theoretical SHANSEP line & correlation Ball cone resistance, source: POV-M (2017)

Conclusion

Vacuum consolidation trials utilizing two different vacuum techniques were conducted in Schardam, the Netherlands. The trials have demonstrated that both vacuum consolidation methods are effective for enhancing the shear strength of the subsoil in dike reinforcement projects. It has been confirmed that the application of vacuum pressure alone leads to a significant increase in strength.

The SHANSEP method has proven to be effective in accurately determining the strength gain. Both field and laboratory tests have indicated that the increase in pre-consolidation pressure and the undrained shear strength is permanent. Furthermore, the increase in the clay layers was more notable than in the peat layers.

Based on these compelling results and conclusions, vacuum consolidation is anticipated to be utilized as an innovative technique for specific sections of the dike in the "Markermeerdijken" project.

Other information about this case

Location and layout

The trial area, located near Schardam, consisted of two sections designed to test the two systems described. The selection of the area was based on a preliminary site investigation. Each section measured 17.5m x 17.5m. Figure 3.98 provides an overview of the area. It's worth noting that the phreatic line at the trial site was in close proximity to the existing surface level of NAP -0.8m at NAP -0.9m.

Figure 3.98 Overview of the area

Site investigation

Prior to the installation of the vacuum systems, a comprehensive site investigation campaign was conducted. This campaign involved the use of Cone Penetration Tests (CPTs), boreholes, and Ball penetrometer tests.

The CPTs were utilized to determine soil properties, layering, and to verify the presence of sand layers. The data obtained from these tests was also used to determine the appropriate length of the BeauDrain-S drain and tube.

Boreholes were performed to obtain soil samples from various cohesive soil layers. These soil samples were then subjected to compression (CSR), Direct Simple Shear (DSS), and triaxial tests. The compression tests were conducted to determine the initial consolidation coefficient, initial pre-consolidation stress, Over-Consolidation Ratio (OCR), and the SHANSEP parameter

"m". The DSS tests were employed to determine the SHANSEP parameter "S" and the initial shear strength "su" under in-situ terrain stress conditions.
In addition, the Ball penetrometer test, a relatively new test, was also carried out to provide detailed information about the undrained shear strength. This test is similar to a CPT, but it employs a cone tip in the shape of a spherical ball, which is approximately 8-10 times the diameter of a standard cone. The Ball penetrometer test offers improved accuracy and resolution, particularly in soft soils. The undrained shear strength can be estimated by correlating the Ball penetrometer data with laboratory testing results. In this case, the undrained shear strength values obtained from the DSS and CAU tests were correlated with the corresponding values from the Ball penetrometer tests conducted at similar depths.

Installation
Due to the extremely soft soil conditions and the high water table at the site, a specialized working method was implemented during the installation of the drains. The primary objective was to minimize the risk of soil squeezing and stability issues. This approach was necessary to isolate the effect of the vacuum pressure alone. To achieve this, a lightweight working platform, approximately 0.5m thick, was constructed using Vulcanic lightweight flugsand and a light geotextile. Timber mats were placed on top of the working platform to distribute the load of the lightweight hydraulic excavator.

In both sections of the trial area, the vertical drains were installed with a triangular spacing of 1m. The installation depth of the drains extended up to 2m above the Pleistocene sand level, which is approximately NAP -8.7m. This working method and installation technique were specifically designed to address the unique soil conditions and to ensure the desired research focus on the impact of vacuum pressure.


Figure 3.99 Traditional liner vacuum system,
left: vertical drains connected to the horizontal drains,
right: liner in trench surrounding the area.

In the section where the traditional liner vacuum consolidation technique was applied, an overlength of vertical drains was utilized. These vertical drains were connected to the horizontal drains. Subsequently, a trench was excavated, and the liner was placed below the water table and backfilled accordingly, as illustrated in Figure 3.99. The installation process proceeded smoothly without encountering any significant issues.

Monitoring during consolidation

Monitoring equipment

To monitor the progress and effects of the trial, monitoring equipment was installed both within and around each trial area. This included the placement of settlement plates (4 in total, 2 per area), inclinometers (7 in total), and piezometers (14 in total, 7 per area), with 2 of the piezometers installed inside the vertical drains. Figure 3.101 provides an overview of the monitoring equipment.

Figure 3.100 Areal picture

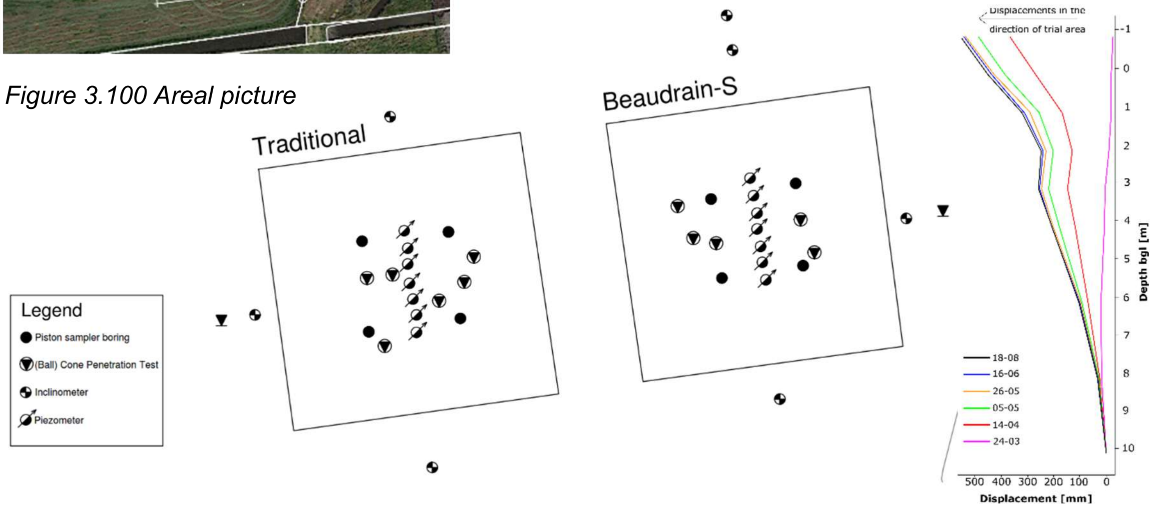

Figure 3.101 Left: Overview of monitoring + test locations, Right: inclinometer readings close to BD-S area

Special precautions were taken to ensure that the installation of the monitoring equipment and cables did not result in any leakage or damage to the liner due to excessive deformations. These arrangements were made to safeguard the integrity of the system. It's worth noting that the Beaudrain-S trial section did not encounter these issues, as depicted in Figure 3.102, where the monitoring equipment remains visible during the consolidation process.

Figure 3.102 Beaudrain-S section with monitoring equipment (after some days of consolidation).

Settlements
Settlement monitoring was conducted continuously in both areas using settlement cells placed on the surface, based on changes in water pressures. Settlement is a crucial indicator of the degree of consolidation in the subsoil. To verify the measurements obtained from the settlement cells, two settlement plates were employed per section/technique. One settlement plate was positioned at the edge of a section, while the other was placed in the middle.

The measured settlements are presented in Figure 3.103. It is evident that the settlements in the middle of the sections exhibit the most significant settlement compared to the settlement plates at the edges. Both systems also demonstrate similar behaviour. Over the consolidation period of 90 days, settlements of approximately 1.1 to 1.5 meters were measured. Following the release of the vacuum (end of June), a noticeable swelling effect was observed over time. The Beaudrain-S area induced a volume change of approximately 210 cubic meters, while the traditional vacuum area saw a change of just over 200 cubic meters.

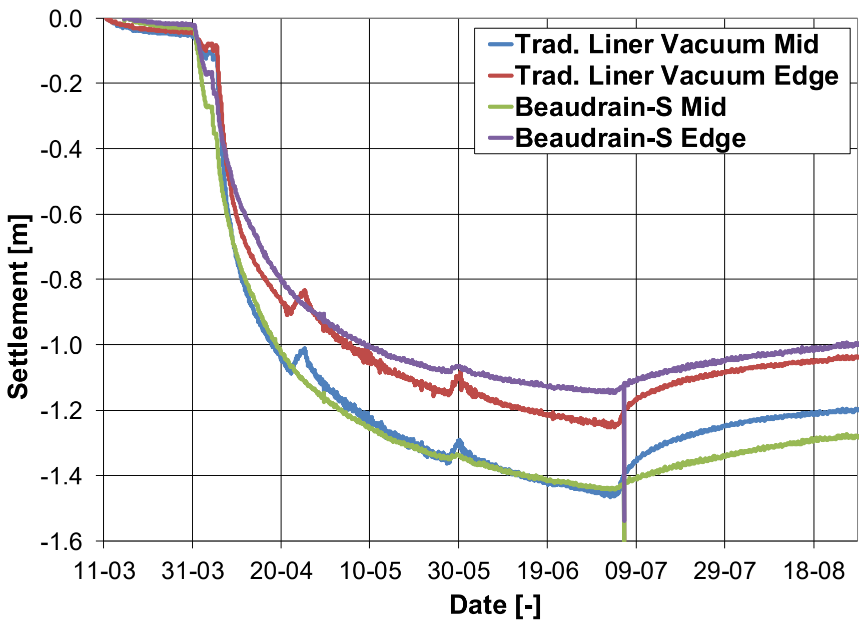

Figure 3.103 Settlements over time during vacuum consolidation

Inclinometers
Inclinometers were strategically positioned at various distances from the vacuum areas to study horizontal deformations both during and after the test, as shown in Figure 3.101. The effect of the vacuum pressure is that deformations occur in the direction of the vacuum area, as there is no driving force from any surcharge. In locations close to the edge of the trial field, horizontal deformations were measured to be approximately ~0.6m.

During the trial, the investigation did not specifically explore whether settlement compensation, which would result in outward deformations, could counterbalance the inward deformations caused by vacuum consolidation. This raises the question of whether settlement compensation could potentially serve as a method to control horizontal deformation, which could be advantageous for the surrounding areas. Further research in this area could provide valuable insights.

Water pressures
Water pressures within the ground were continuously monitored during and after the trial to investigate the time-dependent behaviour of effective stress before and after the vacuum period. This was achieved through continuous measurements using vibrating wire piezometers. The piezometers were distributed across different soil layers and were capable of measuring both positive and negative pressures. Additionally, a reference measurement was taken using a piezometer placed in the permeable layers below the soft soils.

Furthermore, two piezometers were installed inside the vertical drains at the bottom to directly measure the actual vacuum pressure applied to the soil. This pressure differed from the pump pressure due to variations in the height differences between the pump and the water table, as well as flow losses in the system. The pumps maintained a relatively constant vacuum pressure of approximately 90 kPa in the Beaudrain-S area. In the traditional vacuum area, a gradual increase in pump pressures was observed over a two-month period, reaching a stable pressure of 90 kPa thereafter.

The measurements taken in the traditional vacuum area indicated a water pressure drop of approximately 60 kPa in the silty clay layer and around 20 kPa in the peat layer. Similar values were observed in the Beaudrain-S area, with water pressure drops of 60 kPa in the silty clay and 35 kPa in the peat. Figure 3.104 provides an overview of water pressures in the Beaudrain-S area. The water pressure drop in the clay was more substantial in both trial areas and comparable to the water pressure decrease observed inside the vertical drains, which ranged between 60-70 kPa. The slightly lower achieved vacuum pressure in the traditional vacuum area can be attributed to a slower buildup of the vacuum and potential breakdown of the pumps. It is worth noting that the lower water pressure drop in the peat layer was unexpected and may be attributed to the presence of gas in the peat or a different permeability ratio between the vertical and horizontal directions than initially assumed.

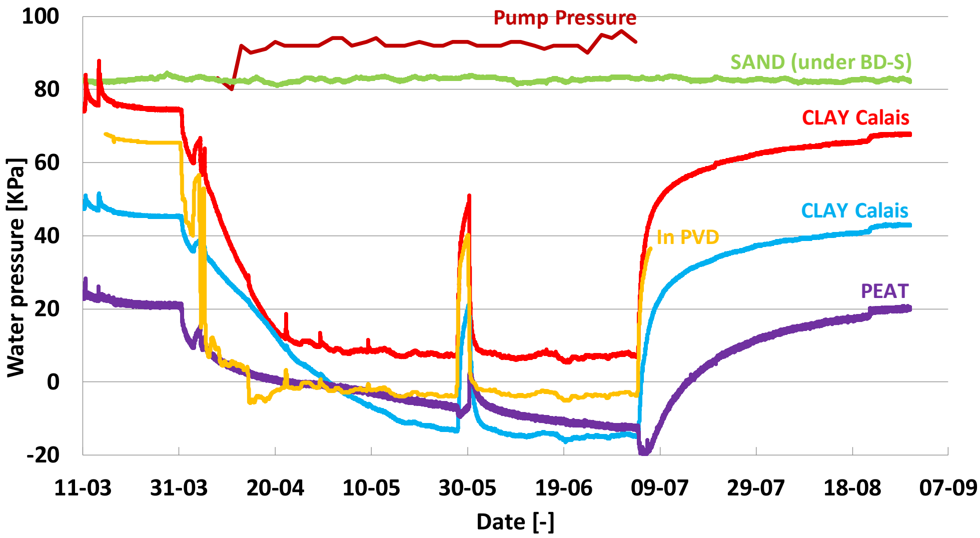

Figure 3.104 Water pressures over time Beaudrain-S area

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3.1.13 Case description: Bamboo-mattress embankment reinforcement at the Semarang-Demak integrated giant sea dyke with toll road, Indonesia (slope stability)

Author: Hendra Jitno, National Institute of Technology (Itenas), Bandung, Indonesia
Keywords: Slope failure; river levees; soft clay; peat; stiff clay; sand

For the corresponding technique factsheet on bamboo mattress reinforcement, please refer to Section 3.2.14.

Setting

The site is located in Semarang, the capital of Central Java, as shown in Figure 3.105. The Semarang–Demak Toll Road is part of Indonesia's National Strategic Projects, as stipulated in Presidential Regulation No. 56 of 2018, which amends Presidential Regulation No. 3 of 2016 on the Acceleration of National Strategic Projects. To expedite the development of the toll road, which is integrated with a coastal sea dyke to address tidal flooding issues in Semarang, the Ministry of Public Works and Housing issued Ministerial Decree No. 355/KPTS/M/2017. This decree authorises the integration of the Semarang City sea dyke with the Semarang–Demak Toll Road project. The toll road is planned to extend 27 kilometers (from STA 0+000 to STA 27+000), divided into two sections: Section 1 (STA 0+000 to STA 10+690) and Section 2 (STA 10+690 to STA 27+000). The most challenging part is Section 1, which is located in a coastal area with very soft to soft soil foundation.

The success of infrastructure development heavily depends on adequate geological data and information. Accurate and reliable geological insights are essential for understanding natural physical conditions, which can either support or hinder a project. Therefore, comprehending the geological context is crucial not only during the planning phase but also throughout implementation and post-construction. By aligning infrastructure with geological realities, developments can be more resilient and environmentally harmonious.

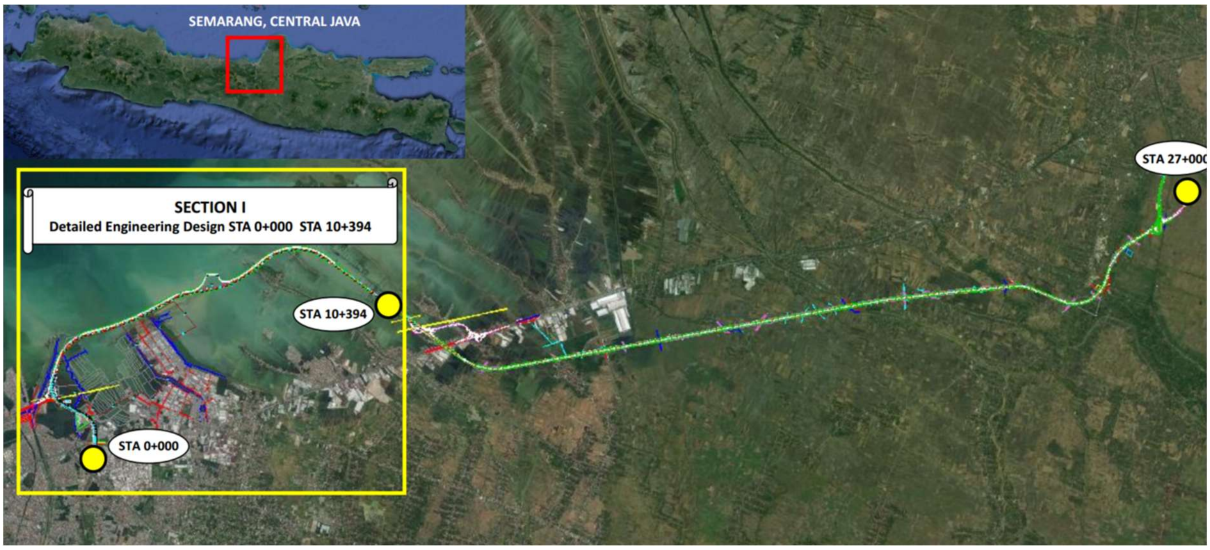


Figure 3.105. Location of the site. The toll route is depicted in the yellow box.

The photos taken from Sections 1 and 2 of the proposed embankment and toll road are shown in Figure 3.106 and Figure 3.107. Figure 3.106 shows the Section 1 coastal area where the embankment is to be constructed, which is the subject of this technical document. Figure 3.107 shows the Section 2 area of the toll road, which will mainly be built as long-span bridges.

Figure 3.106. Swamp Plain Unit in The Northern Part of Section 1 of the Toll Route.

Figure 3.107. Alluvial Plain Unit in Section 2 of The Toll Route.

Problem description

The current problems in the north Semarang area include;

- Severe coastal flooding,

This is often caused by rising sea levels, tidal surges, and land subsidence, leading to frequent inundation of both urban and rural areas. This disrupts transportation, hampers business operations, and interferes with daily life. Additionally, it poses serious health risks due to inadequate sanitation and the accumulation of stagnant water.

- Land Subsidence

In some parts of Semarang, land is sinking at a rate of up to 10–20 cm per year, primarily due to excessive groundwater extraction for industrial and domestic use. This subsidence increases vulnerability to flooding, even in the absence of rainfall, and results in a permanent loss of elevation relative to sea level.

- Urban Expansion on Low-Lying Coastal Areas

Rapid development in flood-prone coastal zones, including informal settlements and industrial estates, is putting more people and infrastructure at risk. This urban sprawl complicates evacuation procedures and hinders effective flood mitigation efforts during emergencies.

- Inadequate Drainage and River Overflow

Rivers like the Banjir Kanal Timur and the Sayung River frequently overflow due to sedimentation, narrow channels, and clogged drainage systems. This results in widespread urban flooding during heavy rainfall and can lead to contamination of freshwater sources with seawater during high tides.

- Tidal Backflow (Saltwater Intrusion)

Rising tides, coupled with the loss of natural resistance from coastal wetlands and mangroves, are causing saltwater to flow inland. This leads to the salinization of agricultural lands and damages freshwater aquifers, threatening drinking water supplies.

- Loss of Natural Coastal Barriers

The widespread deforestation of mangroves for development and fish pond construction has significantly reduced natural defences against waves and tidal surges. As a result, coastal areas face increased erosion and loss of critical habitats.

- Protection of Strategic Infrastructure

Key infrastructure such as the Semarang city center, nearby industrial zones (e.g., Genuk and Sayung), and the Semarang–Demak Toll Road are at growing risk from coastal hazards. There is an urgent need for a comprehensive and integrated solution to safeguard these vital economic and transportation assets.

Why a sea-dyke is needed

A sea-dyke is essential to physically prevent seawater from inundating low-lying areas. When integrated with effective drainage and pumping systems, it can manage inland runoff and bolster urban resilience, supporting long-term economic sustainability in Semarang and Demak.

However, due to a deep layer of soft soil in the area of interest, it is extremely expensive to build a significantly high sea-dyke to manage the problems mentioned above. Thus, some bold innovations are needed to reduce the cost of building the sea wall and transportation infrastructures while achieving the goals to reduce the problems above. This technical document refers to the technology applied at the site to mitigate the extremely high cost of building a high sea-dyke in a deep, very soft to soft soil layer at the north side of Semarang City, Central Java, Indonesia.

Remediation measure(s)

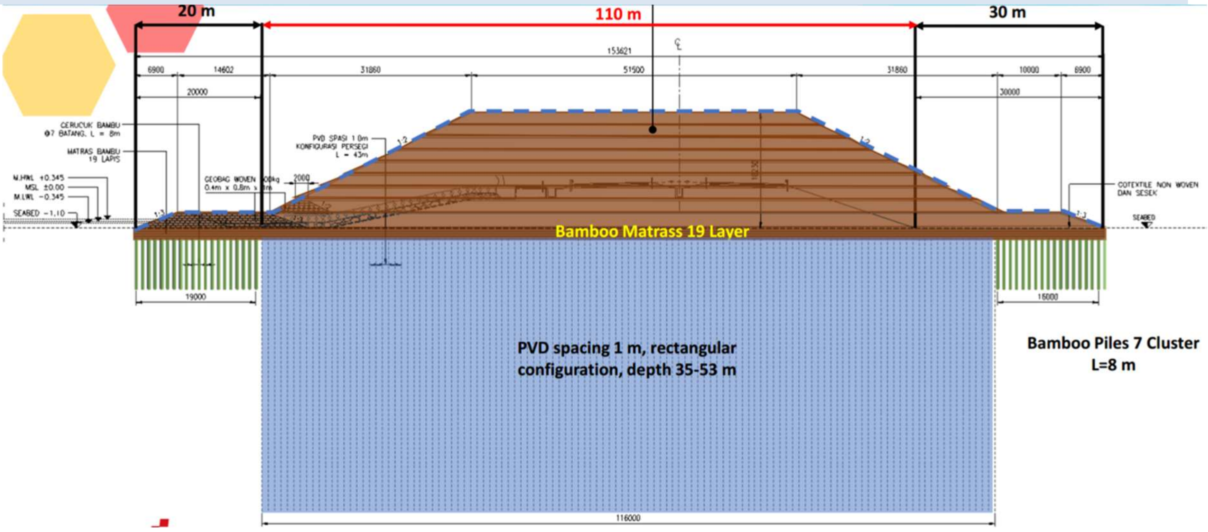


Figure 3.108. Typical cross-section of the embankment

The Section 1 toll road with an overall length of 10.934 km will be constructed using the bamboo-mattress ground improvement technique, combined with Prefabricated Vertical Drains (PVDs) and high-strength woven geotextiles. This technique reduces the cost by 40% compared to the original cost using a conventional modern ground improvement technique, utilizing a much deeper PVD and vacuum consolidation method.

The work commenced in 2022 and is anticipated to be completed by 2027.

Figure 3.109. Installation of high-strength woven geotextile and PVD after installation of the bamboo-matress reinforced fill.

Other information about this case

The design life for the sea dyke is 35 years, which is sufficient to accommodate the expected sea level rise in this area of 0.5m over the next 35 years, due to climate change, and the expected land subsidence of 2.6m.

Environmental loads such as wind and waves are based on a 1,000-year return period to determine the crest elevation of the embankment. This high return period is chosen due to the vital and sensitive nature of the structure, requiring a high level of design safety.
For wave loading, a 10,000-year return period is used to determine the dimensions of the armor layer. This conservative approach is adopted because adjusting or repairing the armor layer after construction is considerably difficult.

The design of the sea dyke was undertaken by PT LAPI ITB, Bandung. Like many other large projects in Indonesia, the design was reviewed by a panel of external experts from different engineering fields. The author was a member of the expert panel in geotechnical engineering.

3.1.14 *Case description: Vertical drain system in embankment body along the Tisza River (slope stability)*

Authors: Edina Koch, Richard Ray (Széchenyi István University), General Directorate of Water Management
Keywords: slope sliding; river levees; high plasticity clay; silt; sandy silt; silty sand; increasing hydraulic loads (intensity or frequency) due to climate change.

For the corresponding technique factsheet on vertical drain system in embankment body, please refer to Section 3.2.15.

Setting

Hungary has one of the extensive river dike systems in Europe along the two major rivers (Danube and Tisza) and their tributaries. There are 12 regional Water Directorates responsible for flood protection. Covering an area of 157,186 km², the Tisza River Basin is the largest sub-basin of the Danube River Basin. The Tisza River is the longest tributary of the Danube (966 km) and has the second-largest flow after the Sava. The Lower Tisza District Water Directorate is state-owned water management institution responsible for operating and maintaining facilities in Hungary's Lower Tisza Valley drainage basin. The directorate's main activities include flood prevention and control, drainage and mitigation of inland excess water, provision of irrigation water, and the sustainable management of surface and groundwater resources.

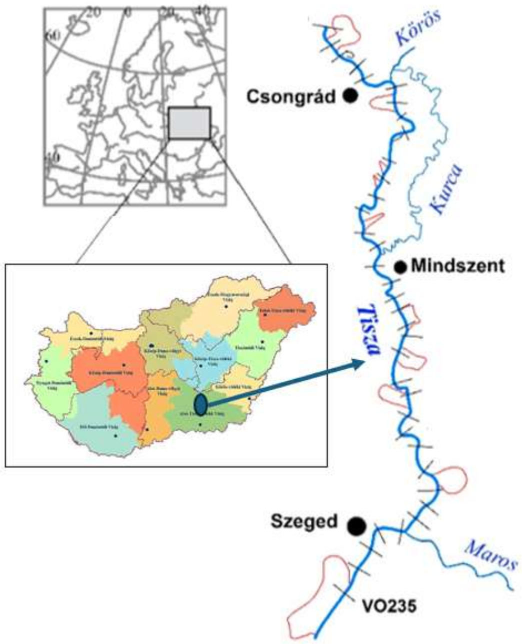


Figure 3.110 Location of the investigated case

The Lower-Tisza Region Water Directorate (ATIVIZIG) protects ~330km long primary dyke:

- ~ 200 km along the Tisza River
- ~ 35 km along the Hármas-Kőrös
- ~ 95 km along the Maros River

ATIVIZIG has five river basins, three of which extend into neighbouring countries, resulting in shared embankments with Romania and Serbia.

The investigated case is a rural levee. The height of the original levee was ~3 m, but it was raised and widened due to the increasing flood level. The measured levee's current height is 6.4 m, with a base width of ~60 m and a 6.3 m wide crest (Figure 3.111).

The landside slope of the levee is 1:4, and the waterside slope is 1:2.5. The foundation and the embankment core are high-plasticity clay, and while the underlying strata are composed of sand and silty soils. (Figure 3.111).

The typical flood event lasts about 3-4 weeks.

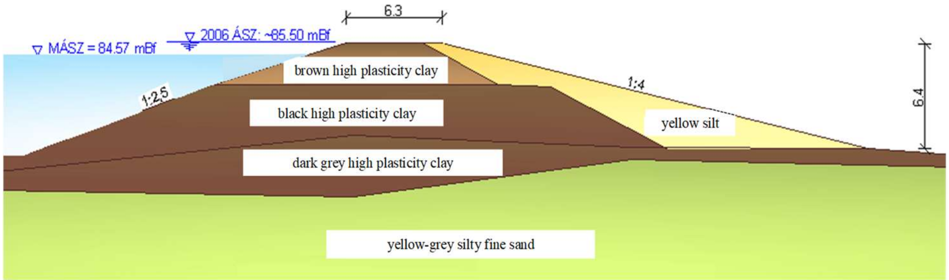


Figure 3.111 Soil profile of the investigated case

Problem description

The case is a slope failure induced by the April 2006 flood near the Tisza River. The failure occurred on the landside of the levee and required immediate stabilization by a sandbag revetment at its toe. During the peak flood, longitudinal cracking occurred at 2/3 of the height, and the landside slope moved ~1 m downwards. The length of the failure was ~80 m (Figure 3.112)

Before the flood, the levee was in good condition. Observations of field performance pointed to the apparent failure mechanism where seepage helped to generate reduced effective stress and strength. The slope moved just as the water level peaked, rising 1.5 m in about a week. The water seeped over the lower, less permeable black clay material and through the upper, slightly more permeable brown zone. This process weakened the strength of the black clay interface and, to some extent, the brown layer. At the same time, seepage gradients almost parallel to the downslope direction increased the sliding forces.

Geotechnical finite element modelling helped determine failure behaviour after the sliding and before reinforcement to focus repair operations.

Figure 3.112 Slope sliding and temporary reinforcement

Remediation measure(s)

Description of the selected remediation methods

After the flood, repair crews reinforced the failed slope using a gravel drain on the landside for long-term stabilization. The coarse-grained drainage system provides seepage control along the landside levee toe. For a schematic cross section including the gravel drain, see Figure 3.113.

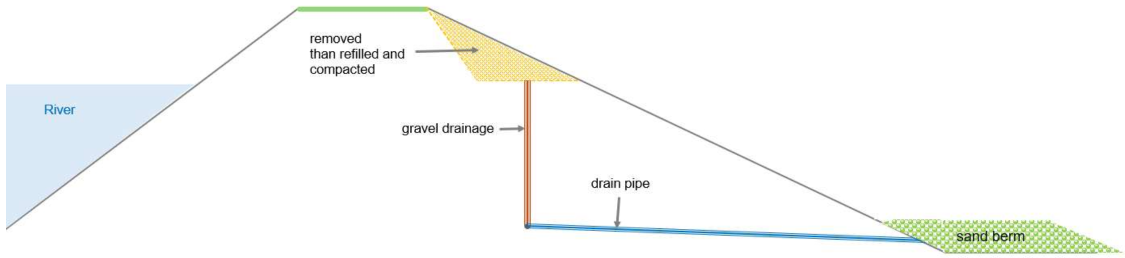


Figure 3.113 Schematic cross section of reinforcement containing vertical gravel drainage

It is a common practice when property and resource limitations restrict the extension of the landside slope.

Performance

The reinforcement of the levee occurred after the 2006 flood. Since then, several floods (with similar levels) have inundated the region, but there have been no problems with the reinforced levee. The drainage system using granular material has provided an effective method for levee stabilization.

Other information about this case

Due to the extreme flood event in 2013, engineers have raised the design flood parameters. The event has prompted new values for design water levels, evaluations of new levee designs, and reinforcement of existing levees.

3.1.15 Case description: Retaining by means of soil nailing along the Zuid-Willemsvaart canal (slope stability)

Authors: Jan Couck (Flemish Government, Geotechnics Division, Belgium), Robin Lievens (Flemish Government, The Flemish Waterways, Belgium)
Keywords: Slope sliding, canal, soil nailing, rainfall induced slope sliding

For the corresponding technique factsheet on retaining by means of soil nailing, please refer to Section 3.2.16.

Setting

This case concerns the restoration of an unstable slope along the Zuid-Willemsvaart canal in Lanaken, Belgium. In March 2020, part of the embankment became unstable on the left bank of the canal between house numbers 61 and 95. The embankment had cracks and subsidence. The location is indicated below:

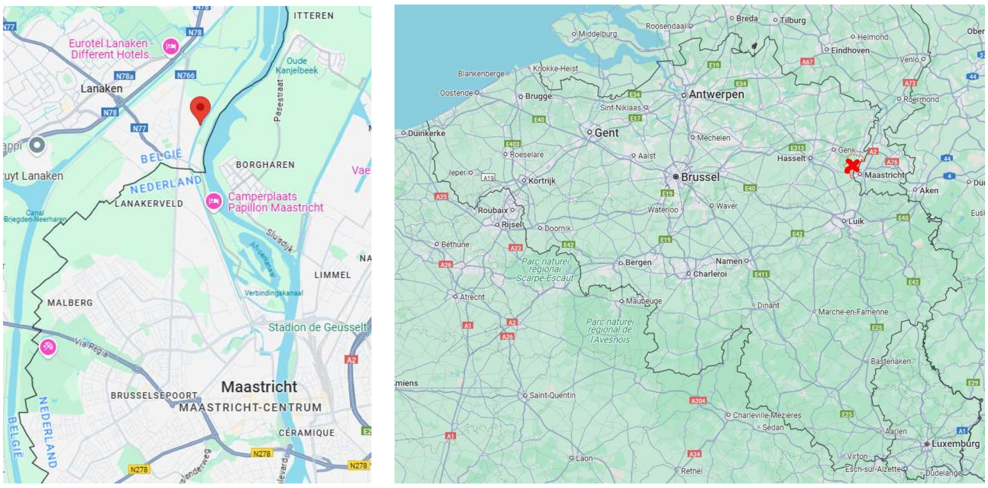


Figure 3.114 Case location

Twenty-five CPTE-soundings were performed along the slope to investigate the subsurface. The subsurface consists of sand. Stones are present in sand layer B. Below is a representative CPT-sounding showing the ground layering:

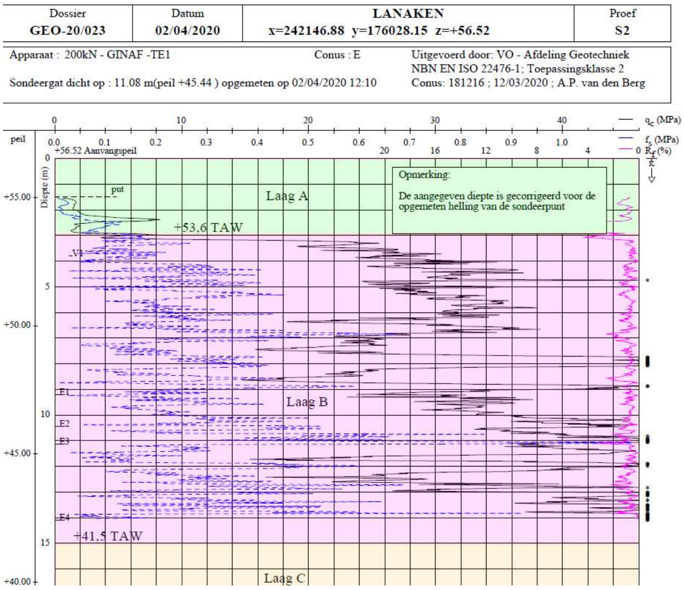


Figure 3.115 CPT sounding

The slope is part of a deep cutting leading to a man-made canal. See the photo below:

Figure 3.116 Picture of canal Zuid-Willemsvaart

The slope is very steep with a maximum of 45°. The slope has a significant height, with a vertical height of up to 17.6 meters from the bottom of the canal to the ground surface. Below is a cross-section:

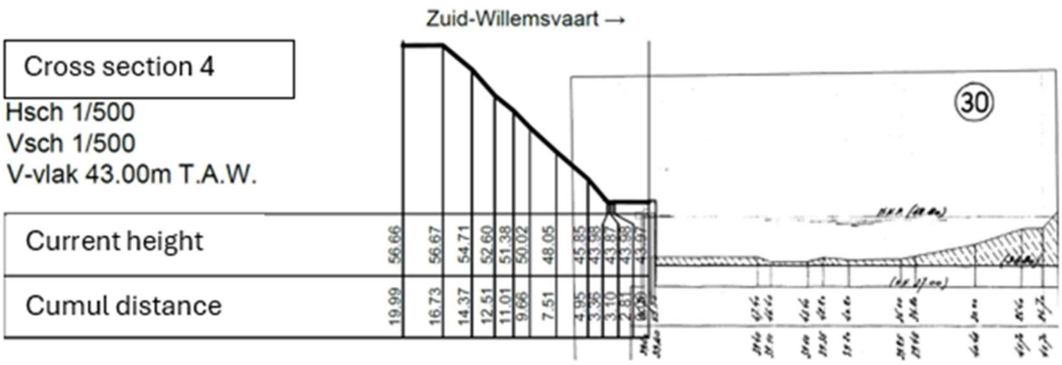


Figure 3.117 Cross section

The original slope was covered with tall shrubs and trees, whose roots reinforced the superficial layer. After clearing this vegetation and following rains, a slope slide occurred. Several houses are located about 7 meters from the crest of the slope and are at risk of sliding.

Problem description

Some pictures of the sliding:

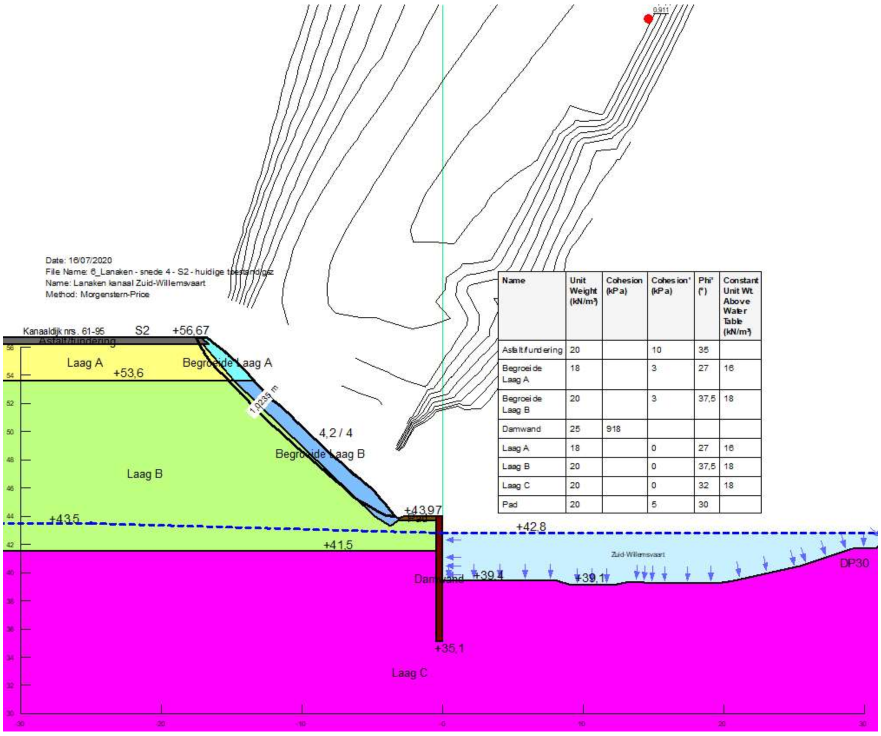
Figure 3.118 Pictures of the slope sliding.

Subsidence and cracks formed at the crest between the roadway and the slope:

Figure 3.119 Pictures of subsidence and cracks near the roadway

A slope calculation indicates that the original slope was nearly unstable, with a safety factor of less than 1.

Initially, the slope was restored by applying a layer of sand cement after removing the displaced soil. Later, it was decided to reinforce the slope to a definitively stable situation, and this in the shear zone and the steepest nearby zones.

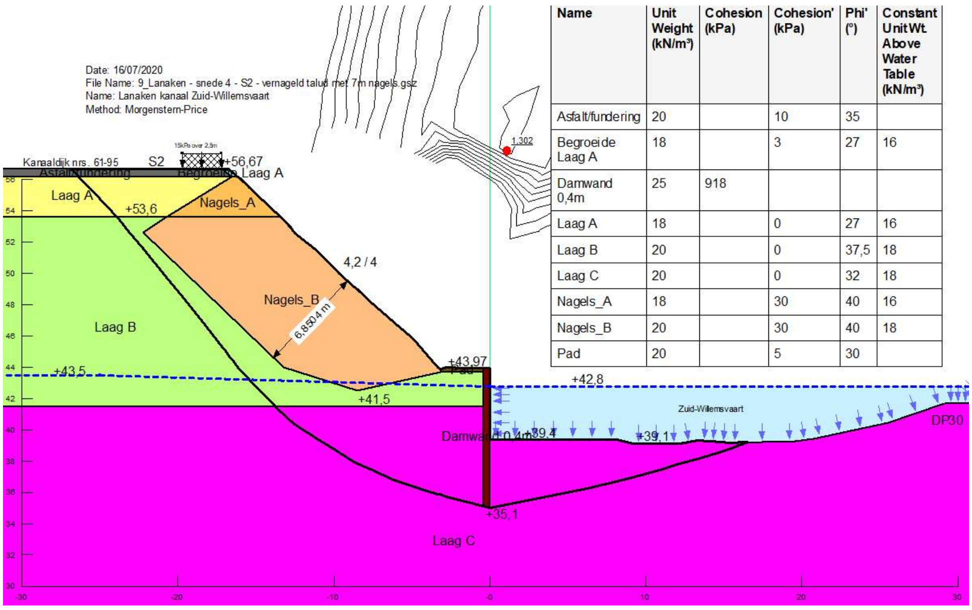


Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m³)
Asfaltfundering	20		10	35	
Begroeide Laag A	18		3	27	16
Begroeide Laag B	20		3	37,5	18
Damwand	25	918			
Laag A	18		0	27	16
Laag B	20		0	37,5	18
Laag C	20		0	32	18
Pad	20		5	30	

Figure 3.120 Stability calculation

Remediation measure(s)

As a remediation measure, it was decided to strengthen the slope with nailing. This has many technical advantages but is expensive. The external stability of the nailed slope was calculated in Slope/W from Geostudio.



Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m³)
Asfalt/fundering	20		10	35	
Begroeide Laag A	18		3	27	16
Damwand 0,4m	25	918			
Laag A	18		0	27	16
Laag B	20		0	37,5	18
Laag C	20		0	32	18
Nagels_A	18		30	40	16
Nagels_B	20		30	40	18
Pad	20		5	30	

Figure 3.121 Stability calculation with nailing in Slope/W

The nails were dimensioned with Plaxis-2D.

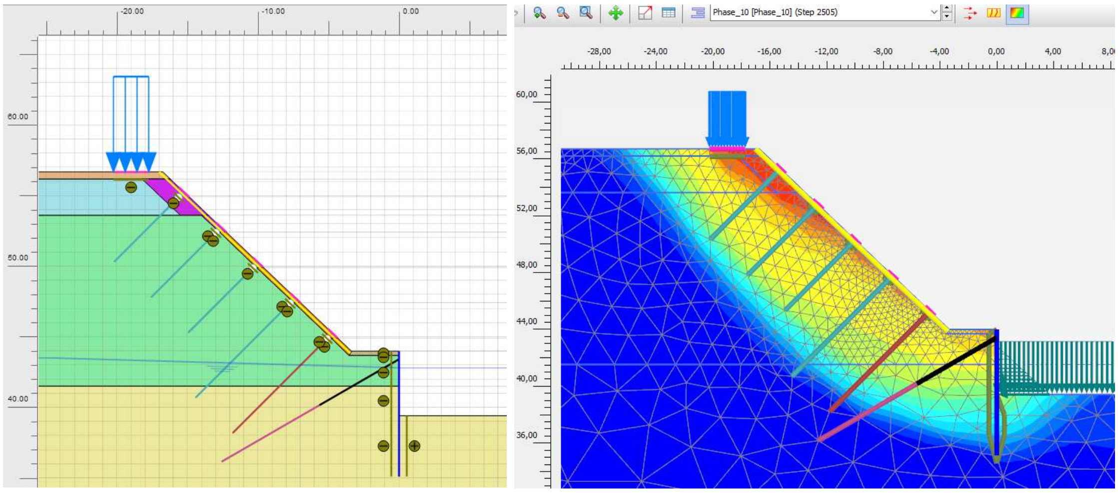


Figure 3.122 Stability calculation and nail dimensioning in Plaxis

The execution of the nailing took place in 2023-2024. Nailing is a common measure in Belgium, but less often applied to canal slopes.

Additionally, the drainage of the road was improved to avoid water entering the slope.
After the nailing, there were no longer any issues with the slope.

Other information about this case

The upper road did not have proper drainage, causing rainwater being collected on the top of the embankment and penetrating into its core. This caused surface erosion.

Execution of the maintenance of the vegetation on the steep and high slope is not easy. Therefore, consideration has been given to the type of vegetation (grass, shrubs, or trees) to be planted on the restored slope. After research and consideration, it has been decided to sow grass.

There are no major fluctuations in the water level in the canal. The canal is deeply dug into the landscape, so there is no risk of flooding the surrounding area.

3.1.16 Case description: Mitigation strategies (drainage and piles) for water induced landslides in fill embankments (slope stability)

Authors: Yonggang Cheng, Yuze Sun, Jianfeng Wang, Hong Kong

Keywords: Embankment fill slope; drainage interception; rainfall-induced landslide; a water intercepting blind ditch

Setting

Water, despite its seemingly gentle nature, possesses the potential to inflict substantial damage on geomaterials over an extended duration. This phenomenon illustrates that in specific scenarios, the weak can overpower the strong and the soft can triumph over the hard. In the realm of geotechnical engineering, water wields considerable influence. Therefore, the resolution of water-related issues should take precedence when addressing geotechnical engineering challenges. Effectively tackling these water concerns is crucial to ensure the long-term durability of geotechnical projects. When addressing water-related problems, it is imperative to adhere to the objective laws of development and harmoniously coordinate human efforts with geological conditions through drainage projects. This cooperative approach significantly enhances the safety and cost-effectiveness of water damage mitigation initiatives.

Effective drainage plays a critical role in the retrofitting and reinforcement of levees, which are vital structures designed to protect communities from the devastating impacts of flooding. Levees act as barriers, preventing water from overtopping and inundating surrounding areas during periods of high water levels or heavy rainfall. However, even well-designed levees can be susceptible to failure if proper drainage measures are not in place. The accumulation of excess water or seepage within the levee system can exert additional pressure on the structure, weaken its stability, and increase the risk of breaching or erosion. In the retrofitting and reinforcement of levees, the assessment and improvement of existing drainage systems are critical. This involves evaluating factors such as the adequacy of existing channels, optimizing their capacity, and ensuring proper maintenance to prevent blockages and obstructions.

Priority should be given to water management in slope construction, particularly in the context of fill engineering. This is especially critical in embankment projects, as inadequate drainage of the fill area can result in groundwater deterioration. This compromises the physical properties of the slope, leading to water pressure and subsequent landslides within the embankment. Therefore, whether during construction or damage control, the drainage project should be of utmost importance to ensure high efficiency and long-term stability. Neglecting the drainage interception project during the construction phase poses a hidden danger to the safety of the fill embankment in the future. Similarly, disregarding the drainage interception project during the management of faulty projects will inevitably require significant treatment measures to mitigate the issues.

The road embankment fill slope has a height ranging from 8 to 12m, featuring a design slope ratio of 1:1.5 to 1:1.75. It is located within a "skip" shaped slope section that corresponds to a natural slope gradient of approximately 22°. To safeguard the slope surface, arch berms and grass are utilized.

The road is located on a lengthy downhill stretch. In the rainy season, substantial rainfall triggered a landslide, estimated to be around 50,000 m³ in volume. This occurrence led to the collapse of the inner graben slopes and blockage of the roadbed drainage ditch. Consequently, surface water accumulated and seeped into the road embankment.

The landslide exhibited a characteristic horseshoe shape, with the back edge of the slope where the embankment is situated deviating by approximately 3-5m. The main axis of the landslide measured approximately 70m in length and 90m in width, while the average thickness of the landslide body was about 10m.

After the landslide occurred, the technicians promptly initiated emergency unloading. Following the temporary stabilization of the landslide, anti-slip piles were strategically installed at the first level platform to provide calculated support. Specifically, Φ2.5m, 22m long anti-slip piles made of C30 concrete were utilized with a spacing of 5m. These anti-slip piles offer an anti-slip force of 983 kN/m.

Figure 3.123 Embankment at the Long Downhill Section of the Route

Figure 3.124 The Horseshoe-shaped Morphology of the Trailing Edge of an Embankment Landslide

Figure 3.125 Landslides Protruding from the Embankment Area

Figure 3.126 Landslide Leading Edge Bulge

Problem description

Based on the engineering geological conditions, the natural slope on the inner side of the roadbed takes the form of a "dustpan" shape, resulting in a large catchment area. Additionally, as the road is situated in a longitudinal slope section, heavy rainfall can lead to blockages in the drainage ditch, allowing a significant volume of surface water to infiltrate the embankment. Consequently, water seepage occurs along the original embankment fill and the underlying mudstone interface, leading to the deterioration of geotechnical properties and the generation of water pressure. Therefore, the primary focus for the permanent engineering treatment of slope damage should be the interception and drainage of the groundwater source, specifically the surface water within the inner side ditch of the roadbed.

Remediation measure(s)

Based on these findings, it is recommended to construct a water intercepting blind ditch in the lower section of the side ditch. The blind ditch should have a depth of approximately 3.0m and a base that extends at least 0.5m below the strongly weathered mudstone, which serves as the water barrier. This approach effectively prevents later surface water from seeping into the embankment. Additionally, drawing from the successful experience of Wang Gongxian's drainage projects in landslide management, it is advised to increase the internal friction angle Φ of the landslide surface parameter by 1-1.5° (a conservative value of 1° can be utilized). This adjustment elevates the slip surface characteristics by one grade, thereby reducing the downward force of the landslide and optimizing the project's support scale. Furthermore, this measure promotes the long-term stability of the project.

Furthermore, the initial scheme places the anti-slip piles solely on the first level platform without considering the resistance prior to the piles. As a consequence, this arrangement necessitates larger anti-slip piles to support the increased thrust from the landslide, resulting in an unfavourable impact on the engineering economy. Additionally, the structural calculation of the anti-slip pile reveals that the original scheme employs Φ2.5m anti-slip piles made of C30 concrete with a spacing of 5m for support. However, through calculations, it is determined that the technicians unsafely obtained a landslide thrust of 983KN/m.

Considering this, to prevent potential overtopping of the sliding body, it is advisable to adjust the placement of the anti-slip pile to the anti-slip section. Furthermore, it is crucial to consider the pre-pile resistance and soil pressure appropriately. By doing so, it becomes possible to effectively reduce both the landslide thrust that needs to be supported by the anti-slip pile and the required length of the anti-slip pile.

After implementing the aforementioned optimizations and adjustments, the required support from the anti-slip pile for the landslide sliding force is approximately 450 kN/m, which accounts for only 45% of the sliding force in the original scheme. Consequently, the engineering support program can be significantly reduced. Building upon this achievement, 13m long anti-slip piles made of C30 concrete, with a pile diameter of Φ2.2m and a spacing of 5m, are strategically installed at suitable locations along the slope's base to effectively provide support for the landslide.

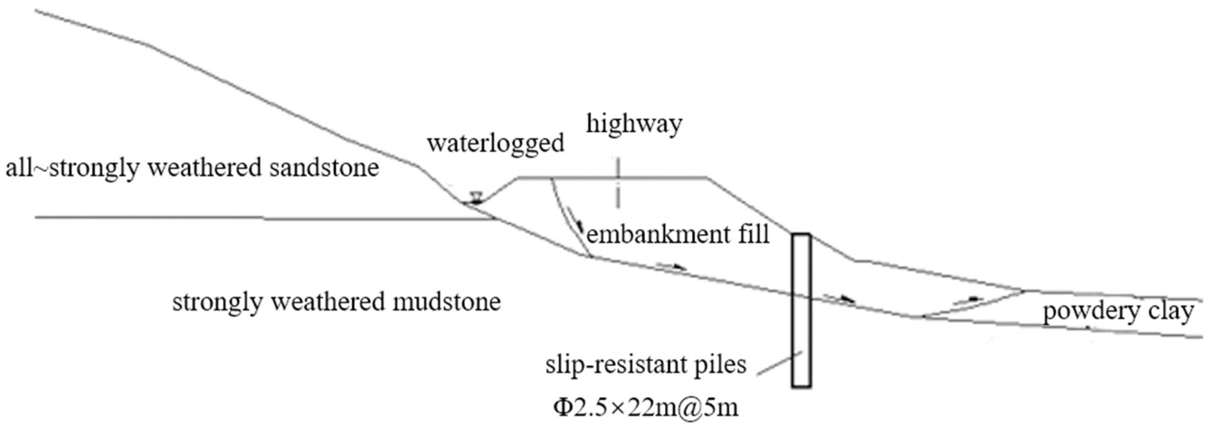


Figure 3.127 Proposed Engineering Geological Sections

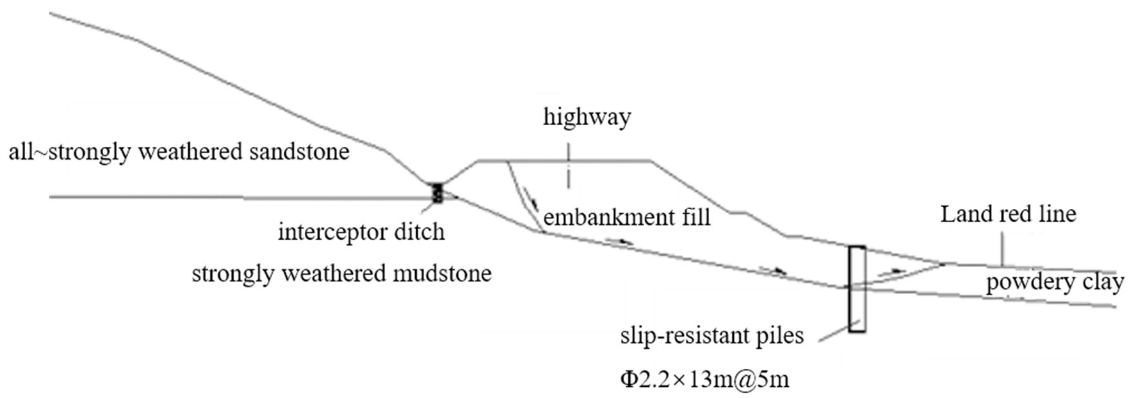


Figure 3.128 Optimization of Engineering Geological Section

The aforementioned case highlights the significance of implementing effective drainage measures and proper pile placement to mitigate water-induced landslide damage. By prioritizing water management in slope projects, we address the issue at its source and adhere to the laws of nature. This approach enhances the efficiency and durability of the works. Furthermore, it is essential to design human engineering projects that leverage natural elements, thereby minimizing the construction scale of the project.

Publications

Mitigation strategies for water-induced landslides in fill embankments (2023). Retrieved from: mp.weixin.qq.com/s/YFN3HFeI725xIK4msPIS3Q (in Chinese)

3.1.17 Case description: Anchored sheet pile walls along the Waal River (slope stability)

Authors: Theo Stoutjesdijk, Esther Rosenbrand and Meindert Van, Deltares, the Netherlands
Keywords: Slope sliding; river levees; soft clay; peat; stiff clay; sand; increasing hydraulic loads (intensity or frequency) due to climate change.

For the corresponding technique factsheet on anchored sheet pile walls, please refer to Section 3.2.17.

Setting

In the central part of the Netherlands the operational area of Waterboard Rivierenland is found. This consists of parts of the provinces Zuid-Holland, Gelderland, Noord-Brabant and Utrecht, a total area of 201.000 ha. Rivierenland is responsible for the primary dikes along the rivers Lek, Nederrijn, Waal and Maas with a total length of 507 km. The dike stretch Gorinchem Waardenburg is found on the north side of the river Waal between Gorinchem and Zaltbommel in Figure 3.129 (see the orange dotted line).

Figure 3.129 Operational area of Waterboard Rivierenland including dike stretch Gorinchem-Waardenburg (orange dotted line) (Source: https://www.waterschaprivierenland.nl/werkgebied)

In this part of the Netherlands the subsoil typically consists of layers of peat, soft clays and stiffer clays for the first approximately 6 meters below the surface. In the eastern part of the dike stretch there is less peat present. Below 6 meters from the surface the Pleistocene sand is found. The subsoil has a low bearing capacity, and uplift and backward erosion piping (BEP) are common phenomena.

The existing dikes before reinforcement typically have no berms and have relatively steep slopes (1 on 2 was not an exception for ancient dikes). After reinforcement the slopes are typically 1 : 2,5 tot 1 : 3 and have berms on the inside of the dike. According to the current safety standards slope stability of the inner and outer slope, safety against piping and crest height are often insufficient. In the dike reinforcement of 23 km length, 17 km will be executed in soil reinforcement and 6 km using constructive solutions.

The dikes along the dike stretch Gorinchem Waardenburg have a history from the year 1200 AD onward. The dikes are rich in historical and cultural values, some parts are densely populated and they cross areas which have high landscape and natural values. Impressions of the dike are given in the following photographs.

Figure 3.130 Photographs of the dike stretch Gorinchem Waardenburg, the Netherlands, (Source: http://terinzage.gralliantie.nl/projectplan-waterwet/)

The existing dikes are mainly constructed in clay. The height of the dikes is 7 to 8 meters above the hinterland.

A typical high water event lasts for about a week.

Problem description

Following the guidelines from 2017 the dikes have been subjected to a thorough safety assessment, in which different failure mechanisms were analysed. Over almost the entire length of 23,5 km the dike does not meet the current safety standards. The main issues are backward erosion piping, slope stability of the inner and outer slope and crest height.

Remediation measure(s)

Of the 23,5 km dike to be reinforced about 17 km is an improvement using soil works as the only technique. If the surrounding space allows this it is usually the cheapest solution, and fits in well with the landscape as we know this in the Netherlands along the rivers.

For a length of about 6 km structural measures were necessary. Usually the lack of space as a result of existing buildings or natural and ecological values are the reason for these more costly solutions. The type of structure that was chosen ranges from a single sheet pile wall, an anchored sheet pile wall, a deep slurry wall or even cofferdams. These techniques are all part of the common practice for dike reinforcements in the Netherlands.

An example of a schematic cross section containing an anchored sheet pile wall is shown in Figure 3.131.

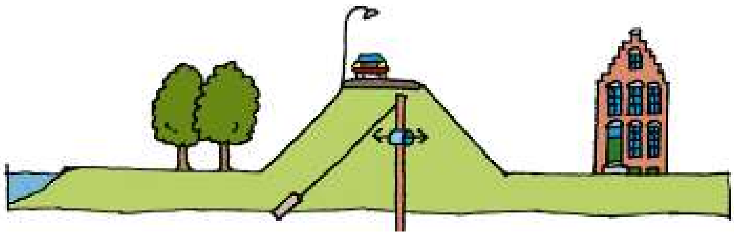


Figure 3.131 Schematic cross section of a dike with an anchored sheet pile wall

The work has been started in 2021 and is expected to be completed in 2026.

Other information about this case

The design period for soil solutions is 50 years. For structural measures this is 100 years. In the design extra hydraulic loads due to climate change (higher flood discharges and waves) and subsidence of the surface in the polder and the crest of the dike have to be taken into account.

The design was iterated in a cooperation between the waterboard Rivierenland, contractors Heijmans, GMB and de Vries & van de Wiel and engineering company Royal Haskoning DHV.

3.1.18 Case description: (Drainage) berm and sheet pile walls along the Adige River (slope stability)

Authors: Alessia Amabile, Alessandro Tarantino, Fabio De Polo

Keywords: reinforcement berm; sheet pile; seepage reduction

Setting

The embankments on the Adige River in the North of Italy were built at the end of the 19th century to straighten the river path. The traces of the ancient meanders are still visible along the alluvial valley and are easily recognizable from aerial photographs and satellite images (Figure 3.132). The Autonomous Province of Bolzano is responsible for the management of about 100 km of embankments along the Adige River.

(a) (b)

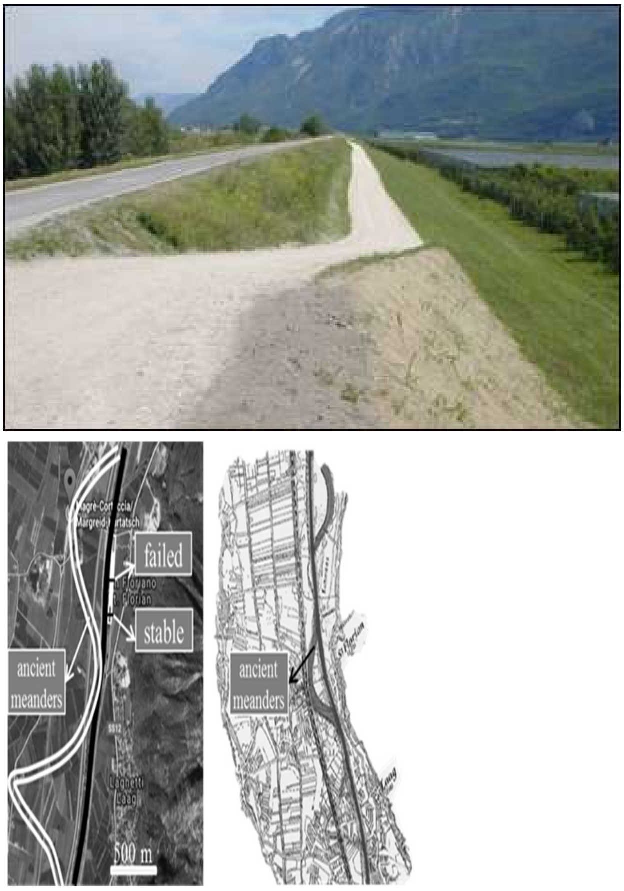


Figure 3.132 (a) Adige river embankment and (b) ancient meanders of the riverbed before the construction of the embankments. Image courtesy of Casa Editrice Athesia/Tappeiner (Werth 2003).

The embankments were built using the natural alluvial materials available on site, which consist of gravel and coarse sand in some areas, and finer, silty sand in other areas. The foundation layer is constituted of similar materials, with a shallow foundation made mostly of a sandy material, with significant coarse fraction of alluvial origin, which corresponds to the ancient riverbed where the Adige River used to flow before being straightened. The deeper foundation layer is a sandy deposit with local lenses of finer material, with significant organic fraction in some areas, which corresponds to the glacial deposit where the Adige River formed its meandering path.

Problem description

Over the years several segments of the embankment have experienced failure or have shown signs of incipient instability of the landside slope, such as boiling and water springs a short distance from the landside toe. Water springing at the toe tends to be clear, which indicates that internal erosion is not likely to be an issue.
Numerical modelling validated with monitoring data from piezometers and tensiometers has shown that a typical flood event of the Adige River, lasting between 15 and 20 hours, is long enough to generate considerable seepage into the embankment (Amabile et al., 2020). Given the relatively coarse grain size of the embankment material, this flood duration can lead to a significant increase of pore water pressure in the core and at the toe of the embankment.

Remediation measure(s)

The measures reviewed here were applied to a ~30 km long segment between Bolzano and Salorno. Of the 30 km segment in need of remedial measures, about 9 km on the right-hand side and 14 km on the left-hand side of the river were reinforced with a berm on the landside slope, with or without a drainage filter in place.
This was the most cost-effective solution, and fitted in well with the natural landscape. In other sections, a different remedial measure was applied, which consisted of the construction of sheet pile walls in the core of the embankment. This was implemented along about 6 km on the right-hand side and about 4 km on the left-hand side. This more costly measure was adopted in the sections where space was limited, due to the motorway running parallel to the embankment. These techniques are all part of the common practice for embankment improvement in the area.

Technique 1: Toe reinforcement with berm, with or without drainage

This solution has been employed for the largest cross sections, where natural material can easily be sourced, and equipment and machinery can safely access the site for construction and installation. A typical cross section is shown in Figure 3.133. Wherever possible, the berm was built with coarse-grained material so that the hydraulic conductivity of the embankment is not decreased. In the sections where such material was not available, a drainage is installed to ensure water flow and avoid buildup of pore water pressure. Stages of construction are shown in Figure 3.134. When building the drainage, after the excavation (stage 1-2) a geotextile membrane was laid down first (stage 3), followed by a layer of coarse sand and gravel (stage 4-5), which also improves the stability of the slope against settlements. After that, the drainage was covered and the material for the berm was put in place (stage 6). It was important to monitor carefully the vibrations during the transport and movement of materials as they may have an impact on the stability of the embankment.

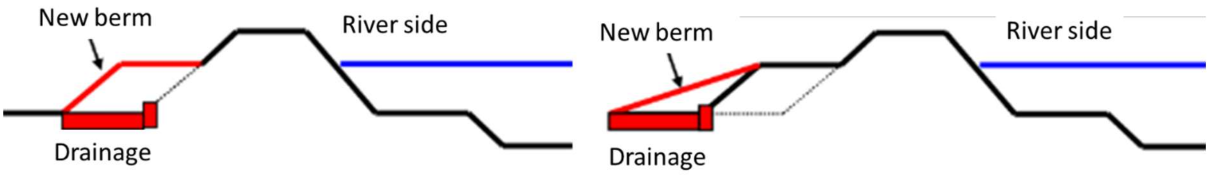


Figure 3.133 Typical cross section after construction of the berm for a) an embankment without previous berm, and b) an embankment with existing berm.

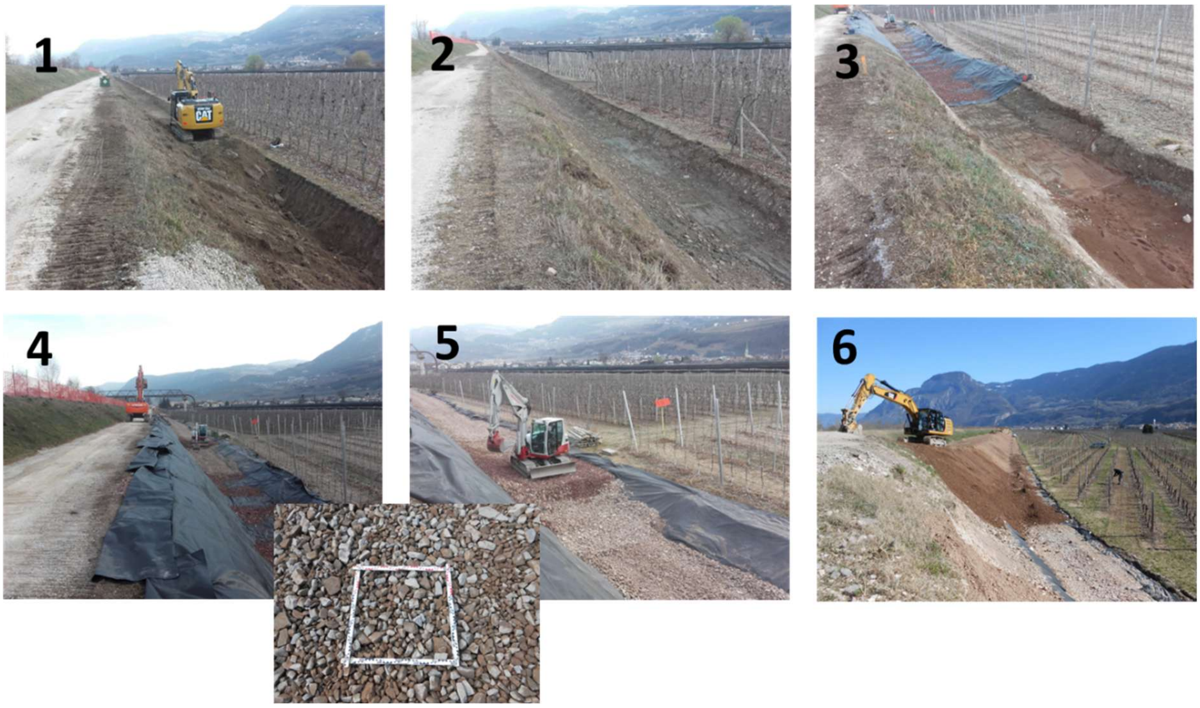


Figure 3.134 Installation steps of filter drainage at the toe of the berm and construction of the berm.

Technique 2: Sheet pile wall in the embankment core

Sheet pile walls were installed to reduce seepage in the embankment in those sections where limited space did not allow for the construction of a berm. A typical cross section is shown in Figure 3.135. Different types of sheet pile walls were installed, including cement-bentonite walls, steel and PVC sheets and concrete walls. Whilst the concrete walls also help with the structural reinforcement of the embankment, the PVC and cement-bentonite walls have the only function of abating pore-water pressures in the embankment and its foundation on the land side by creating an impermeable barrier in the core that lengthens the seepage path from the riverside to the landside. The cement-bentonite walls were installed by jet-grouting the mixture of cement (water to cement ratio ~0.8) and bentonite (3.5 - 4% in weight) as shown in Figure 3.136, while prefabricated sheets were driven into the soil (Figure 3.137).

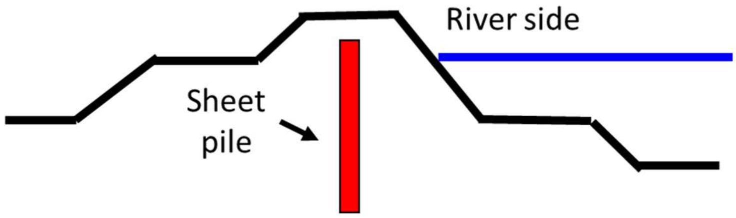


Figure 3.135 Typical cross section after construction of the sheet pile.

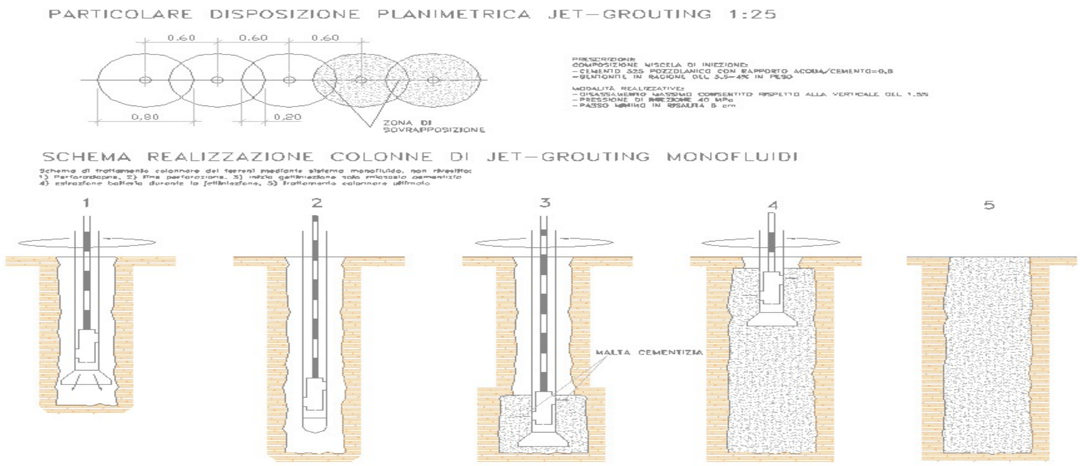


Figure 3.136 Jet-grouting technique for bentonite-cement sheet pile installation.

Figure 3.137 Installation of sheet pile walls.

Performance

The performance of the sheet pile walls has been monitored during flood events recorded in autumn 2023 by comparing the water pressure measured by piezometers on the riverside and on the landside. The Figure 3.138 show that sheet pile walls have performed effectively by reducing the hydraulic head on the landside by about 1.7 m compared to the riverside and by keeping the water level below the ground level.

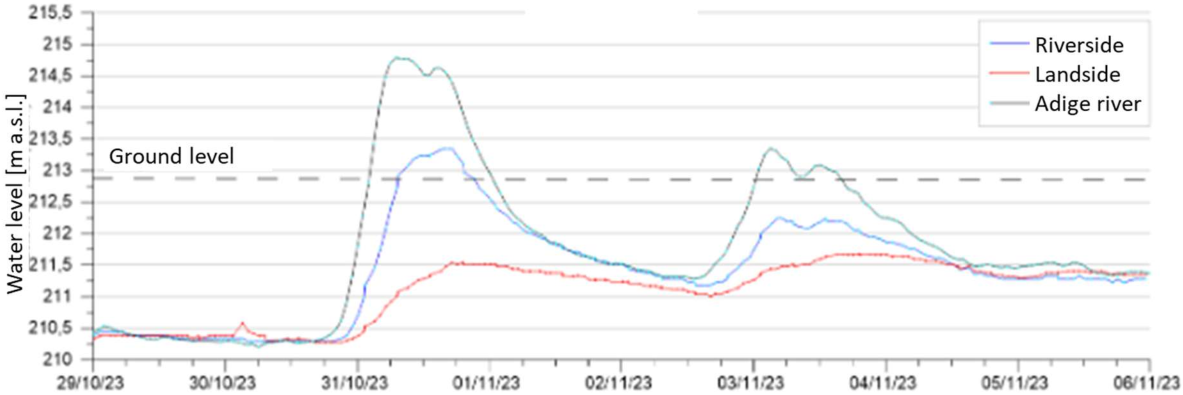


Figure 3.138 Water levels measured during a flood event by piezometers on the riverside and on the landside of the diaphragm.

References

Amabile, A., Pozzato, A., Tarantino, A. 2020. Instability of flood embankments due to pore-water pressure build-up at the toe: lesson learned from the Adige River case study, *Canadian Geotechnical Journal,* 57(12): 1844-1853, https://doi.org/10.1139/cgj-2018-0372.

Werth, K. 2003. Geschichte der Etsch: zwischen Meran und San Michele Flussregulierung, Trockenlegung der Möser, Hochwasserschutz. Casa Editrice Athesia/Tappeiner, Bolzano, Italy.

3.1.19 Case description: Anchoring and nailing system 'JLD Dike stabilizers' at the Ringdijk Watergraafsmeer Amsterdam (slope stability)

Authors: J.A. Teeuw, H.T.J. de Bruijn

Keywords:

- Failure mechanisms: Slope (in)stability.
- Type of setting: A levee in a crowded city
- Subsurface characteristics: soft clay and peat.
- Additional design considerations or uncertainties for design: Very little room te inforce the levee

Setting

The Ringdijk Watergraafsmeer (hereinafter "Ringdijk") lies within the Water Authority 'Amstel Gooi and Vecht' management area. The Ringdijk is part of the regional inland water barrier along the Ringvaart canal of the Watergraafsmeer in Amsterdam. A section of the Ringdijk was rejected in 2012 due to insufficient macro-stability of the inner slope (hereinafter "stability"). The section is located between Wibautstraat and Middenweg (hereinafter "project location"). The section to be reinforced is approximately 600 meters long. The AGV water board was tasked with increasing the stability of the Ringdijk. Due to the road and buildings behind it, the available space for dike reinforcement is very limited.

Figure 3.139 project site in Amsterdam

Problem description

During the periodical assessment of the levees in Amsterdam the 'Ringdijk van de Watergraafsmeer' didn't meet the requirements.

The available space at the project site is insufficient for traditional dike reinforcement in the ground. Therefore, innovative dike reinforcement techniques were also included in the alternative study. One of the promising alternatives was the 'JLD Dike Stabilizer'. The 'JLD Dike Stabilizer' is a dike reinforcement method that actively reinforces the dike internally. The waterboard decided to use this technique.

Remediation measure(s)

Description the selected remediation method

The JLD Dike Stabilizer is an active stabilization system. When the JLD Dike Stabilizer is installed, prestressing is applied to the system. Due to subsurface creep, this prestressing decreases over time. To ensure the stability of the water barrier, a lower limit and a signal value for prestressing have been established. When the signal value is reached, the system must be re-tensioned.

The project was an innovative pilot application. In figures below a description of the method is shown.

The JLD Dike Stabilizer is an anchoring and nailing system that can increase the strength of the subsoil and provide resistance to shearing, thus increasing the dike's stability. The JLD Dike Stabilizer consists of five components (see Figure below):

1. Anchor;
2. Tie rod (connects the anchor to the end plate);
3. LDE (fin element that slides over the tie rod);
4. LDP (end plate to which the tie rod is attached near ground level);
5. Tension bolt (connects the tie rod to the end plate).

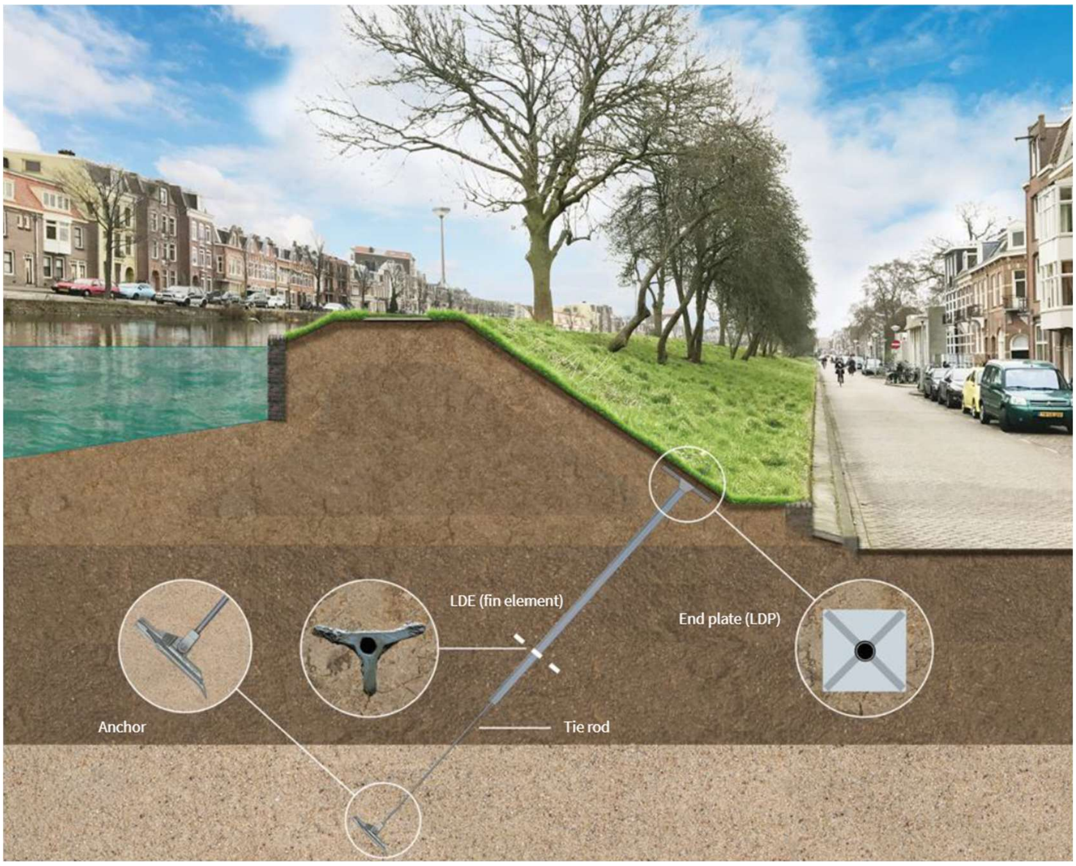


Figure 3.140 Anchor components overview

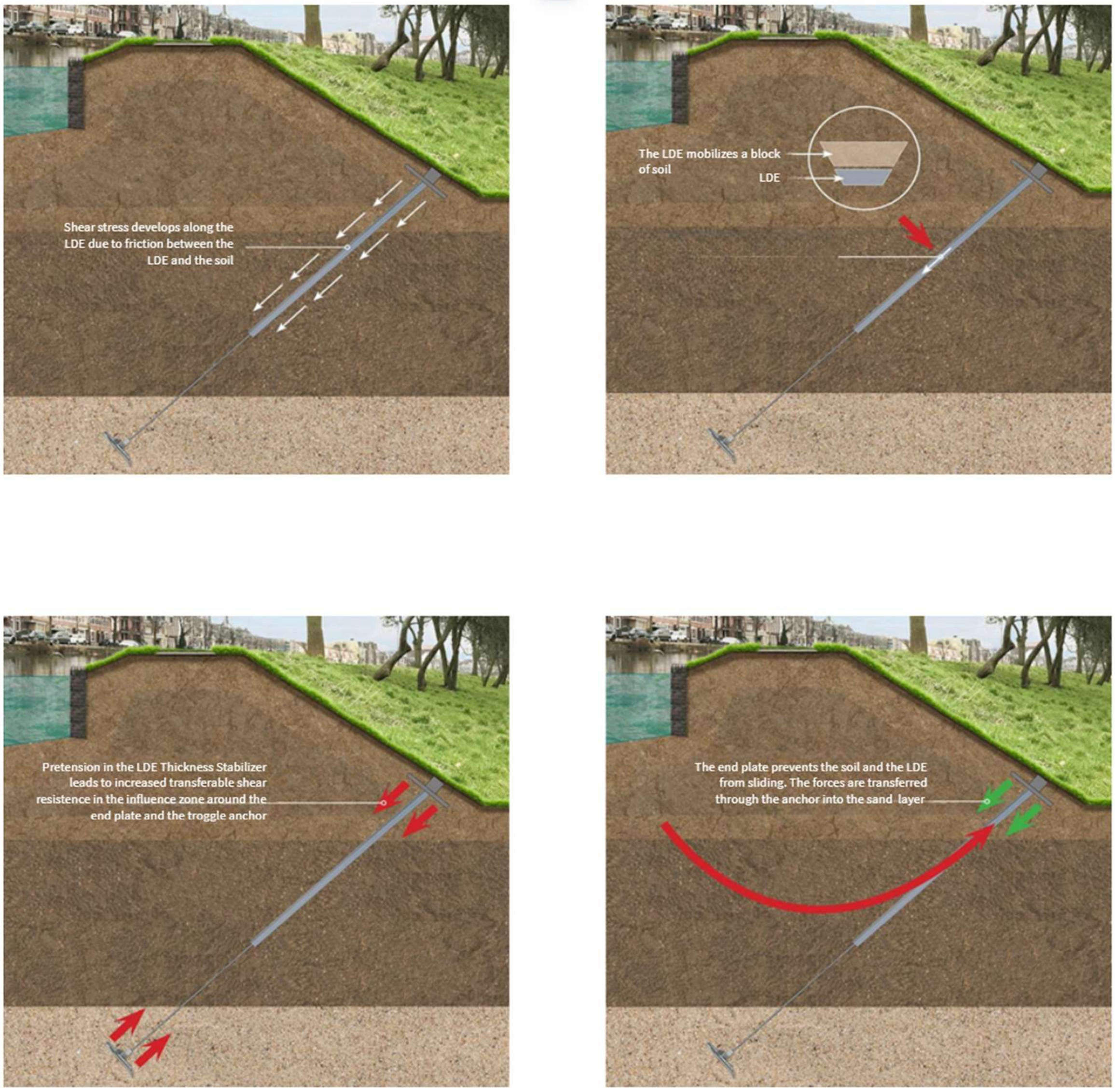


Figure 3.141: Mechanism of the JLD Dike Stabilizer

An anchor is inserted into the ground at an angle on the inner slope and driven into the ground at depth using a driving rod. The anchor is then unfolded and anchored in a deeper (sand) layer. This allows a tensile force to be transferred to the subsoil.

A screw connection connects the spring-loaded anchor to the tie rod. The driving rod pulls the tie rod along with it to the desired depth during installation. The LDE element is then pressed into the ground over the tie rod. The installation method is therefore soil displacement. The LDE element will rest on the anchor rod under lateral load.

At ground level, the tie rod is attached to the head plate by means of a tension bolt and a nut. The steel tension bolt is screwed over the tie rod and extends through the head plate. The head plate is attached to the LDE but is not structurally connected. By applying tension to the head plate with a nut over the tension bolt, the tie rod is tightened and preload can be applied. The

LDE will be pressed slightly further into the ground. Since the LDE can slide freely over the tie rod, it will not be significantly preloaded.

Performance

The implemented technique worked quite well, the only downside was the collection of the tension in each anchor in the pressure logger box. In the pilot the contractor used wireless methods for transferring this data. That proved to be failure-prone. For future systems to be installed a wired data collection system would be better.

Design manuals/standards/codes of practice

The method is described in “POVM Vernagelingstechnieken”; ISBN: 978-90-829248-1-7; © POV Macrostabiliteit, 2019

3.1.20 Case description: River levee stabilization with soil mixing along the Scheldt River (slope stability)

Authors: Leen Vincke, Leen De Vos, Daniel Verastegui; Geotechnical Division of the Department of Mobility and Public Works, Flanders, Belgium

Keywords: Slope sliding, river levees, stiff clay

For the corresponding technique factsheet on soil mix walls, please refer to Section 3.2.18.

Setting

Along the right bank of the river Scheldt in Belgium several slope instabilities exist between the towns of Melle and Dendermonde. These slope instabilities are all related to the presence of a Paleogene stiff clay. The clay, a Merelbeke member of the Gentbrugge formation, shows high plasticity and a rather low residual shear strength.

Figure 3.142 Location overview Scheldt (Source: https://www.geopunt.be/)

Some of the instabilities are quite shallow and are limited in extent, while others are much larger, extending up to a distance of 100 m from the river, and reaching a depth of 15 m, depending on the depth of the clay layer at the respective location.

As shown by monitoring of inclinometers installed at some of the instability locations, large masses of soil are continuously shifting towards the river at a rate higher than 2 cm/year. Most of the movement occurs in the winter season when the low tides in the river coincide with high groundwater tables. Common stabilization methods such as toe berms, terracing, stabilizing piles and sheet pile walls proved insufficient in some cases so a less conventional technique (in Belgium) was evaluated.

The present case history refers to the successful application of soil mixing shear walls to stabilize a river bank at the town of Melle. A similar project in the Wichelen area is planned for execution in 2026.

Problem description

At a few locations the levees along the Scheldt are unstable, slowly sliding towards the river at a higher rate than 2 cm/year. Sliding occurs within a high plasticity stiff clay layer present in almost all the area. This gradual sliding results in damage and associated risks as the river levees are often in close proximity to roads or other infrastructure (e.g. houses and industry). Figure 3.143 below show the location of the site in the town of Melle where stabilization works were executed at the end of 2020.

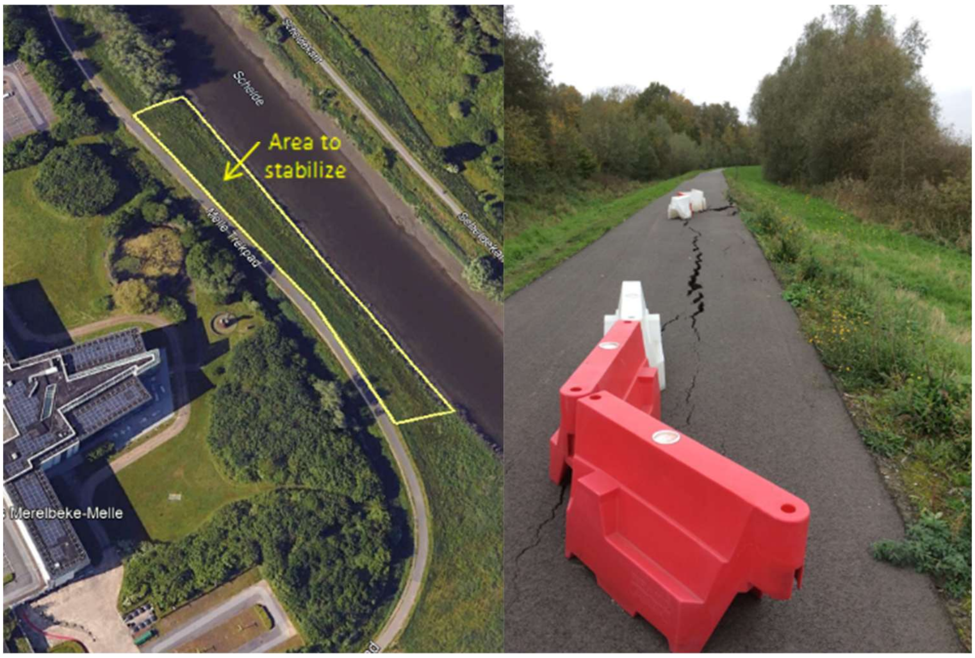


Figure 3.143 Slope instability

Remediation measure(s)

Description the selected remediation methods

The soil mixing technique involves mixing a chemical stabilizer such as cement with a natural soil in situ to create column or panel elements with higher strength and stiffness than the original material. Conventionally, soil mixing walls have been successfully used for earth retaining structures (often in combination with anchors).

Here soil mixing walls oriented perpendicularly to the river centerline have been used to stabilize the levee. In this way the soil mixing walls act as shear walls, more efficient than isolated soil mixing columns as shear walls are not subject to the same type of bending actions that isolated columns do experience.

Figure 3.144 illustrates the configuration of the soil mixing walls at Melle. Soil mixing walls (with a length = 9m, height = 8m, thickness = 0.55m) were installed with a center to center spacing of 5.5m.

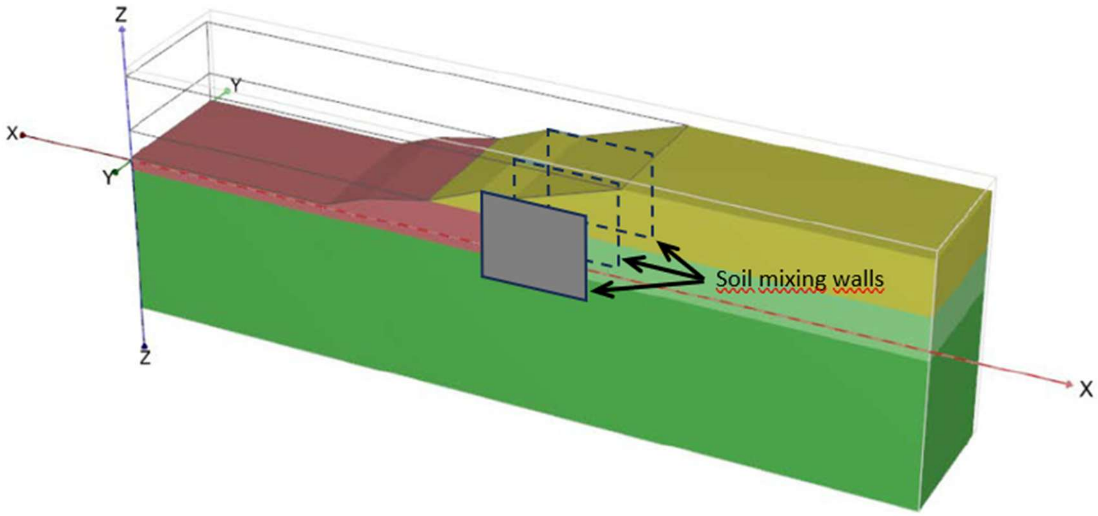


Figure 3.144 Soil mixing walls

A positive point of the soil mixing wall stabilization is that it does not interfere with the groundwater/river interaction. Furthermore, in this particular case it was an economic alternative to other more conventional solutions such as stabilization with sheet pile walls or soil nailing.

However, the technique presented also challenges. Soil mixing in the stiff clay layer (essential for the success of the technique) was difficult. The cement type and water/cement ratio of the slurry need to be carefully assessed through laboratory testing in combination with trial soil mixing panels if no previous experience is available. Another challenge but perhaps not unique to this technique was eliminating or minimizing any sliding during the installation works but also during hardening of the cemented material.

Performance

After the installation of the soil mixing shear walls, horizontal displacements at the site have been monitored to evaluate the performance of this stabilization method. Figure 3.145 below illustrate relative horizontal displacements close to the ground surface within the site before and after stabilization. The striking difference of horizontal displacement proves that the stabilization with soil mixing walls was successful and that the levee at Melle regained stability.

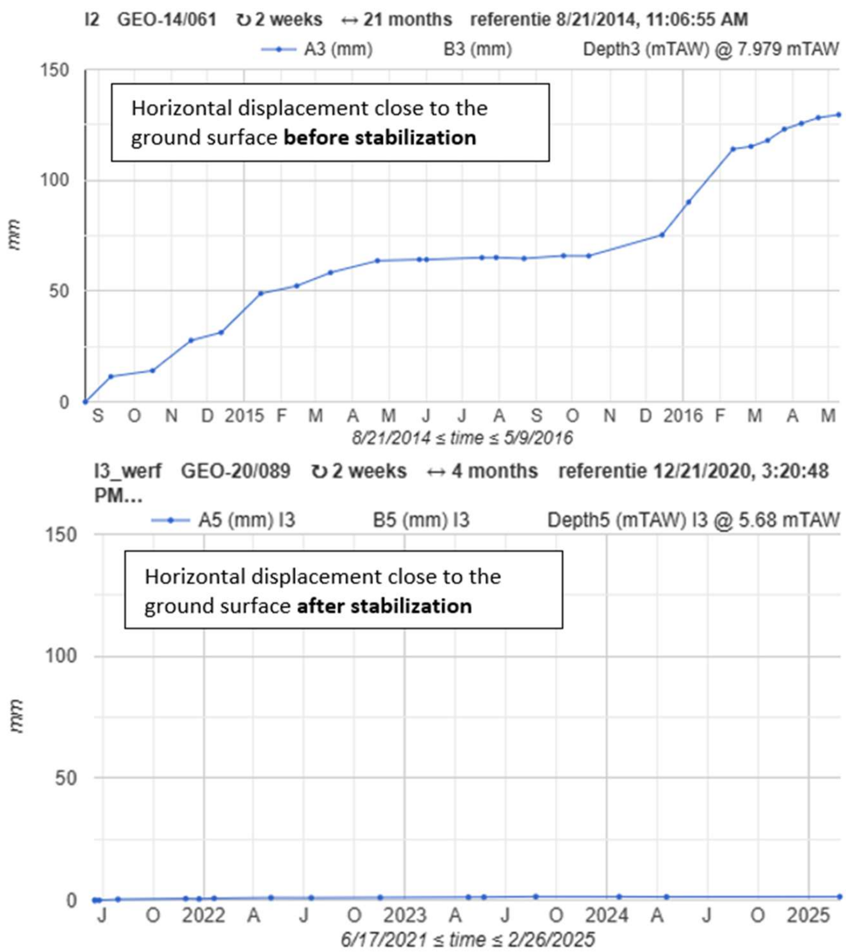


Figure 3.145 Displacement before and after remedial meausure.

Other information about this case

Monitoring of horizontal deformations and groundwater table during the installation proved crucial as it allowed to adapt the stabilization works to minimize sliding and increase safety.

Technique factsheets

Contributions by members of the
Technical Committee on Geotechnical Aspects of Dikes and Levees
(TC201)

3.2.1 Technique factsheet: Relief wells (internal erosion)

Authors: Nicholas Bidlack, USACE, John "Ben" Tatum, USACE, Nicholas "Nikko" Aleman, USACE

For the corresponding case study, please refer to Section 3.1.1.

Failure mechanism(s)

Backward erosion piping through the foundation.

Description technique

The primary goal when design relief wells is to reduce seepage uplift pressure and vertical gradients in the foundation to a safe level, thus lowering the probability of initiation and progression of failure modes like backward erosion piping. Relief wells provide a filtered and controlled exit for underseepage flow.

The site characterization is more critical than the specific analysis method chosen. Thorough field and geologic studies are essential to define the foundation strata, including the thickness and characteristics of the pervious aquifer and the overlying fine-grained blanket. Identifying geologic heterogeneities like abandoned channels that can significantly impact seepage and well performance is crucial. Pilot holes are typically drilled near well locations to verify foundation conditions and inform filter pack and screen design. Obtaining accurate survey data around the site is equally important since floodside topography/bathymetry and protected side topography can impact seepage entry and exit conditions, respectively.

The most commonly used analysis method is the infinite line approach. This historical method, used for most USACE wells since the 1950s, assumes an infinite line of uniformly spaced wells parallel to a linear seepage source. It is based on Blanket Theory, which models the foundation as a two-layer system. The design uses well factors and equations derived from theoretical and model studies to calculate pressure and flow, aiming to reduce excess head to an allowable level based on safety factors. Head losses within the well system must be accounted for in the calculations. This method often involves iterating parameters like well spacing, radius, and penetration ratio to achieve the design goal cost-effectively. Historical design charts and nomograms were used in this process.

Since real-world well systems have finite lengths, techniques are used to account for end effects, which can increase uplift pressure and discharge.

Applicability

Relief wells are best suited where a fine-grained top stratum (blanket) overlies a more pervious stratum (aquifer). This is particularly effective when the blanket is not thick enough to resist potentially destructive seepage pressures caused by excess hydrostatic pressure in the underlying aquifer. Excess head is present whenever the total head in a pervious aquifer is above the protected side land surface as shown in Figure 3.1. These conditions often exist landward of levees and is the case along a majority of Mississippi River.

Relief wells have little impact on seepage pressures in thick pervious strata where an overlying confining layer is absent on the land side of the levee. In the absence of confinement, they can still capture flow but other alternatives (i.e., land side berms or impervious cutoff walls) are generally more appropriate.

Relief wells are a valuable tool for controlling uplift pressures and safely managing underseepage, particularly where a pervious aquifer is overlain by a less pervious blanket layer, but their effectiveness depends heavily on proper design, installation, site conditions, and ongoing maintenance. They can also be a favourable remediation alternative with site constraints and/or limited rights-of-way are present rendering berms or impervious cutoff walls difficult to construct.

Design

Design criteria

Key Design Parameters and Considerations:

Aquifer Properties: Accurate estimates of aquifer thickness, permeability (k), stratification, and anisotropy are vital, especially for partial-penetration wells. While estimates of uplift pressures depend mainly on relative permeability between the aquifer and the blanket, well flow estimates depend directly on the permeability of the aquifer. In cases where well flow estimates are critical, field measurements of aquifer permeability may be advisable. In other cases, estimates of aquifer permeability derived from empirical correlations with D10 values may be adequate.

Blanket Properties: The contrast between the aquifer's transmissivity and the blanket's vertical permeability and thickness significantly influences pressures. Multi-layered blankets may be transformed into an equivalent single layer for analysis using the process set forth in TM 3-424 and EM 1110-2-1914.

Well Geometry: Design involves selecting the well diameter (typically 6 to 18 inches), spacing, and screen length, and screen slot size. In general, it is more cost effective maximize the diameter and screen length (i.e. aquifer penetration) of each well to reduce the total number of wells. However, experience has shown that total well lengths greater than 120-130 ft are sometimes not cost effective. And wells larger than 10 inches in diameter typically require a borehole greater than 24 inches in diameter to accommodate an adequate thickness of filter pack around the well screen, which can require more specialized equipment and increase costs. Well screens should penetrate the main water-bearing strata for effectiveness. Well diameter should be large enough to accommodate submersible well pumps capable of discharging about 1.5x the design discharge of the wells.

Well Components: Each well typically consists of a filter pack of blended natural sand and gravel to allow water through while preventing migration of aquifer materials, a well screen with open area to allow water to enter, a riser pipe to convey water upward from the screen to the ground surface, and some sort of well head to direct discharge water and prevent surface water from entering the well. Design includes specifying materials for each component. Filter pack gradations may be selected from commercially available well filters or custom designed to maximize open area. Well screens are typically stainless steel wire-wrapped screens for permanent wells or PVC screens for temporary wells. Riser pipes are typically solid-wall pipes of the same material as the well screen. Well heads for permanent wells are often short lengths of stainless steel wire-wrapped screen 2 to 2.5x the diameter of the well itself and fitted with a protective cover. Well heads for temporary wells vary greatly depending on the application. Well heads may be designed to discharge vertically at the ground surface or horizontally below grade into a ditch, pipe, or other low-lying area adjacent to the well. Wells designed to discharge vertically should usually be fitted with a simple check valve to prevent surface water from contaminating and clogging the well when the wells are not flowing. Wells designed to discharge horizontally should usually be fitted with a flap gate for the same reason.

Well Head Loss: Accounting for head losses from water entering the filter/screen and moving through the riser is crucial and impacts calculated efficiency and required drawdown. Well head losses typically fall into 3 broad categories: entrance losses, friction losses, and velocity at discharge. Entrance losses account for head expended traveling through the aquifer, any residual skin or “mud cake” remaining on the walls of the well construction borehole, the filter pack, and the well screen. Velocity losses account for head expended traveling upward through the well screen and riser pipe. And velocity losses account for the height of water built up above the well’s design discharge elevation due to the velocity or “jet” effect as the water flows above the top of the well.

Future Performance Degradation: Design must anticipate declining performance over time due to issues like biofouling or mineralization. Biofouling is the growth of microorganisms on the filter pack and well screen, typically iron-reducing bacteria, that reduce effective open area of well screen resulting in greater well losses. A common historical practice is to design assuming a 20% performance loss, ensuring safety factors are met with 80% of initial effectiveness. Groundwater chemistry influences this degradation.

Tailwater and Discharge: The design must safely collect and discharge the well flow away from the protected structure. Collection systems include collector pipes, ditches, and outlets. Standpipes can be used to raise discharge elevation at low river stages. Wells are categorized by discharge type: D-type (at surface) and T-type (below-grade). Particular care should be exercised to ensure that the well discharge is set as low as practical relative to the surrounding terrain. Ditches or low-lying areas located near the wells can allow seepage pressures in the aquifer to “short circuit” the wells (i.e. exit through the lower ground surface rather than through the well outlet), rendering the wells ineffective.

Risk Considerations: Modern design incorporates risk assessment to supplement deterministic requirements. This involves evaluating Potential Failure Modes (PFMs) worsened by seepage and assessing how design choices influence the probability of initiation and progression of these failures, often represented in event trees. Efficiency values from pumping tests can be used in a risk-based approach for maintenance decisions.

Which factors affect the dimensioning?

Well system geometry involves selecting key geometric parameters:

Well Spacing: Wells must be spaced sufficiently close to intercept seepage and reduce hydrostatic pressure to safe values landward of the wells. Closer spacing generally results in lower excess heads. Designs often start assuming an infinite line of wells and then adjust for finite length and end effects.

Well Penetration: Relief wells should penetrate the principal pervious strata to achieve efficient pressure relief. The concept of "effective penetration" for stratified or anisotropic aquifers is crucial for design calculations. Low penetration (e.g., <25% or 20%) significantly reduces efficiency. In most cases, the screen length of each well should be at least 50% of the aquifer thickness.

Well Diameter: Must be large enough to handle maximum anticipated flow, allow testing and servicing, and limit velocity head loss. Diameters are typically between 6 and 18 inches.

Discharge Elevation: The elevation where water exits the well system impacts well performance. Collector systems must be designed to prevent short-circuiting, prevent backflooding, and maintain desired discharge elevations.

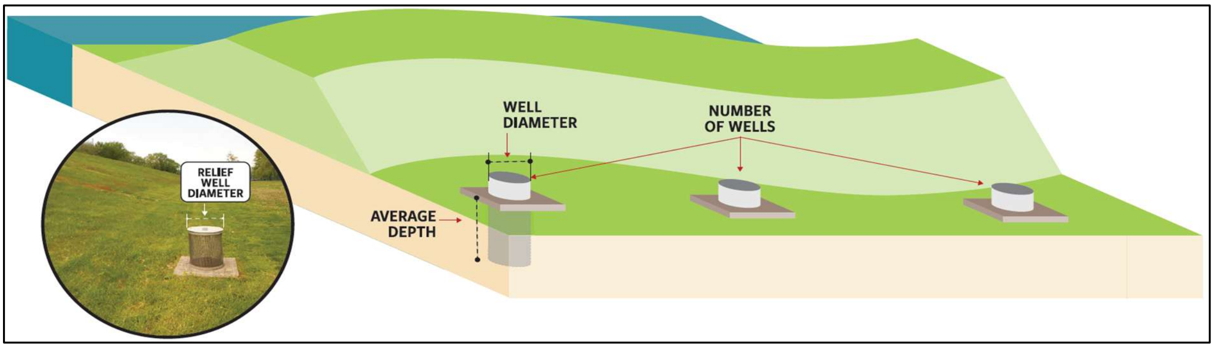


Figure 3.146 - Typical Relief Well Layout

How are key parameters for this technique measured or estimated?

Site characterization (geology, stratigraphy, site features):

Foundation Investigations: Thorough field and geologic studies are essential. These include exploratory borings to define seepage entrance and exit conditions, and determine the depths, thicknesses, and physical characteristics (like permeability) of the top stratum and underlying pervious strata on both the wet and dry sides of the structure. Occasionally, borings should penetrate completely through the pervious strata to determine their full aquifer thickness. Design reaches are developed based on conditions encountered during the subsurface exploration.

Survey Data: Obtaining accurate survey data around the site is important since the presence of landside ditches, riverside borrow pits, and channel bank bathymetry can significantly impact seepage entrance conditions on the floodside and seepage exit conditions on the protected side.

Pilot Holes: Before well installation, pilot holes are typically drilled near well locations. These are sampled continuously or at frequent intervals to verify foundation conditions and check the design parameters like filter pack and screen sizes. Selected pilot holes are extended below the aquifer to confirm its thickness and characterization.

Well and System Geometry (Diameter, Penetration, Spacing):

Diameter: Selected by the designer based on functional requirements (flow capacity, testing/servicing access) and velocity head loss considerations. Borehole diameter is sized based on well diameter plus filter thickness.

Well Screen Length and Penetration (W/D): Selected as trial design parameters. The percentage of aquifer penetration (W/D) is calculated based on screen length and aquifer thickness. For stratified aquifers, an effective penetration is calculated from transformed aquifer properties.

Spacing (a): Selected as a trial design parameter. Designs iterate on spacing, along with penetration and radius, to achieve the target land side head reduction. For finite well lines, spacing near the ends may be adjusted to account for end effects.

Location: Typically, the most advantageous location is at the land side toe of the levee. Occasionally, wells are located at the toes of seepage or stability berms or near a ditch. Final locations may be adjusted in the field based on verification from pilot holes and site constraints. Historically, wells were located at critical seepage spots identified in the office or field. Wells should be located such that they can be accessed for testing, maintenance, and observation during high water events.

Well Component Materials (Screen, Filter):
Selection: Materials (e.g., stainless steel for screen, sand/gravel for filter) are selected based on durability, resistance to groundwater chemistry (informed by water chemistry tests), strength, and compatibility with foundation soils.
Filter Gradation and Screen Slot Size: Designed to prevent infiltration of foundation sands while allowing sufficient flow. Design uses grain size distribution curves and standard filter criteria (e.g., D15/d85 ratios) based on laboratory tests or site samples. Screen slot size is compatible with the filter pack gradation.

Other design considerations

Relief wells are best suited to situations where 1) the prevailing geology consists of a relatively impervious confining layer or “blanket” overlying a more pervious aquifer, 2) land use and real estate limitations make a smaller footprint advantageous, and 3) the additional volume of underseepage water generated by the wells can be handled safely and economically. Wells require more maintenance and generally have shorter useful life than some other methods (e.g. earthen berms). And because they provide a preferential seepage path, they typically increase the rate at which water seeps into the protected area. However, they are compact, and can be designed to fit within a minimal real estate footprint.

Relief well outlets should be protected to ensure proper function. A circular concrete pad is typically cast around the well outlet to prevent erosion due to flow and the concrete can be sloped to promote drainage toward a collector ditch. Flow paths leading toward a collector ditch can be lined with rip rap to prevent erosion of collector ditch side slopes. The concrete pad and well outlet should be protected with steel bollards to aid with visibility and to help prevent damage from vehicular traffic or damage during regular maintenance activities such as mowing.

Construction

Which methods of construction are available?

Permanent wells are typically installed using reverse-rotary drilling methods in which the drilling fluid flows down the annular space between the borehole wall and the drill stem, collects cuttings from the bottom of the borehole, and flows back up to the surface via the drill stem. This method tends to reduce smearing and caking on the borehole walls that could increase well losses.

Other construction considerations?

Using the reverse-rotary installation methods requires that the water level inside the borehole is higher than that at the site. A typical requirement is that the hydrostatic groundwater level is no shallower than 10 feet below the ground surface for well drilling and installation. This helps to maintain hole stability even as tooling and equipment exits the hole or water loss occurs in the aquifer. Drilling fluid additives are typically not used, however, if they are they should be removed. Relief well screen and risers typically come in sections to facilitate easier construction. Threaded pipe connections and welded pipe connections are available and both have been successfully constructed. Centralizers are required to ensure that the minimum filter pack thickness is achieved on all sides of the screen/riser. The filter pack is tremied into the borehole from the bottom up and an impervious seal is constructed above the filter pack to the ground surface.

Development of the well is necessary to help the well obtain maximum efficiency. This typically involves mechanical procedures and chemical treatments to remove finer sand near the well screen and breakdown any drilling fluid additives that may have been used for well installation. Pump testing is typically conducted to obtain a baseline specific capacity and/or efficiency for a well. Pump testing and development should be performed with the groundwater going no lower than the top of the well screen. A sand infiltration test is typically required to test the performance of the well and ensure that the well is developed and the filter pack is not allowing the aquifer sand to pass.

Installation, development, and pump testing activities effect the local groundwater at a site. Particular care should be taken when performing multiple activities at a site as adjacent activities to ensure that adjacent activities do not effect pump test results.

References

Design manuals/standards/codes of practice

Engineer Manual (EM) 1110-2-1914, "Design, Construction, and Maintenance of Relief Wells." This manual, dated March 7, 2025, was issued by the Department of the Army, U.S. Army Corps of Engineers, Washington, DC.

Technical Memorandum (TM) No. 3-424, "Investigation of Underseepage and Its Control, Lower Mississippi River Levees." This document was prepared for the President, Mississippi River Commission, Corps of Engineers, by the Waterways Experiment Station (WES), Vicksburg, Mississippi

3.2 2 Technique factsheet: Deep soil mixing techniques for a cut-off wall (internal erosion)

This contribution is based on: *FICHE TECHNIQUE Sol mixé/mélange en place Deep mixing techniques From Recueil de méthodes et de techniques de confortement et réparation des digues de protection en remblai of the Comité Français des Barrages et Reservoirs (CFBR).*

Main authors: A. LE KOUBY (Univ. Eiffel); L. GERVAIS (DREAL)
Contributors: D. POULAIN (INRAE); M. SUTTER (INRAE)
Reviewers: N. AUGER (DREAL); L. GERVAIS (DREAL); S. LAMBERT (KELLER Fondations Spéciales); L. SAUSSAYE (Cerema); SOLETANCHE-BACHY; S. PATOUILLARD (DREAL); J.-C. PALACIOS (SAFEGE); B. CHALUS (CNR)
Finalisation: A. RULLIERE (INRAE)

Failure mechanism(s)

Relevant failure mechanisms in order of importance:
1. Internal erosion
2. Slope sliding
3. Liquefaction and breach flow sliding (in under water slopes)

Description technique

Definition:

Deep soil mixing techniques used on levees cover a large number of techniques that consist in mixing the soil in place with a binder material in order to modify its hydraulic characteristics (usually with the aim of reducing permeability) or mechanical characteristics (usually with the aim of improving mechanical performance).

This fact sheet presents several techniques and materials used to carry out mixing in place.

Primary functions:

Creation of an impervious cut-off wall in the body of the dike or the foundation, anchored deeply enough to limit seepage that could initiate/worsen internal erosion within the structure and/or its foundation.

Secondary functions:

Support, soil reinforcement.

Required performance:

- mixing the soil in a uniform way to avoid creating areas of weakness, even voids.
- creating a cut-off wall that meets the required permeability and/or strength criteria.

The purpose of these impervious cut-off walls is not to modify the structural strength of the structure but to ensure sealing of the levee and thus to protect it from internal erosion phenomena. To ensure their role in limiting seepage during flood, the impervious cut-off wall run from the crest of the levee to a sufficient depth to avoid the risks of internal erosion.

Applicability

These techniques can be used for almost all kinds of levee geometry, regardless of the environment (river or sea). However, the equipment and materials used sometimes limit this use, in particular because of:

- the workspace needed for the equipment (trencher, drill, injection equipment, etc.);
- the reactivity of the binder material with the environment.

Design

Design criteria

Permeability of the different zones of the levee and foundation. Resistance to the different internal erosion mechanisms of the levee and foundation (mainly suffusion and regressive erosion).

Modelling of internal flow is necessary. Flow velocity after construction of the wall must be inferior to the critical velocity for internal erosion initiation in all areas and in particular at the bottom of the wall.

Which factors affect the dimensioning?

Principle:

The main factor to design such structures is the permeability of the cut-off wall. It is mainly dependent on the nature of the soil, the type and amount of the binder applied, but also on the thickness of the cut-off wall. The depth of the cut-off wall is also an important design parameter, to limit internal erosion processes. The design is carried out on the basis of the geometry of the dike and the characteristics of its foundation and is verified through flow modelling.

Several iterations may be required to optimise sizing in terms of:

- cut-off wall thickness.
- cut-off wall depth (either the position of the foot of the cut-off wall or a substratum-type stop criterion).
- permeability after setting of the soil-binder mixture.

The formulation of the mixed soil should be adapted according to the required objective and is based on formulation studies.

How are the key design parameters for this technique measured or estimated

Required parameters:

The parameters required for design include, in particular:

- the nature of the soil in place (particle size distribution, D_{max}, VBS (methylene blue test), w_L);
- the presence of underground cavities or decompressed areas;
- the expected performance of the soil-binder mixture including setting time;
- the teams' experience with the chosen technique.

Technical specifications:

For this particular technique, an onsite test section is required. It may also be appropriate to request the installation of a site laboratory to take check samples at a defined frequency.

In practice, it is important that the contractor has carried out a formulation study before site work begins. For each treated layer or for the most sensitive layer, results must be obtained for the permeability tests (to check sealing of the soil-binder mixture) and uniaxial compressive strength tests UCS (to check the setting of the mixture) in the laboratory at 7, 28 then 56 and/or 90 days for the chosen formulation (soil types and hydraulic binder(s) and water contents). These criteria must then be checked on the material sampled during work site.

Other design considerations

Site management constraints:

- site accessibility for equipment and supply of anhydrous binder/grout;
- storage area and width of the dike crest;
- expected production rate (overall duration);
- practical considerations in terms of:
    - overhead clearance;
    - restoration of the underground structure;
    - restoration of roadways;
    - traffic constraints;
    - environmental protection;
    - waste storage and disposal;
- methods for using this technique in case of any existing structures.

Site constraints:

- state of compaction of the material in place (assessment of the risk of settlement during cut-off wall production operations);
- average adsorption of the grout/soil-binder mixture by the ground;
- workspace needed;
- level of any water table, or water level of the body of water;
- stability of the dike (slope stability and puncture resistance) including the presence of equipment and materials on the crest of the dike.

Construction

Which methods of construction are available?

Mixing is performed using various tools. In general, tools such as hydromill trench cutters, trenchers or devices derived from the auger (Figure 3.147) are used.

The principle of the technique using a trencher consists in producing an impervious cut-off wall by mixing the soil in place with a hydraulic binder and water. The ground is treated using a rotary tool with a vertical axis, allowing mixing of the soil in place.

The rotary mixing tool is made up of drill rods, transverse blades and pointed auger bits. The lowering and raising of the tool are facilitated by the simultaneous injection of cement grout using nozzles specially placed at the end of the auger in the case of large-diameter columns and also on the mixing blades. The mixing tool ranges from 0.60 to 2.40 m.

(1) (2) (3)

Figure 3.147 Tools for different mixing in place techniques: (1) Cutter Soil Mixing: secant panels (Figure from: Bauer) (2) Trencher: mixed soil trench (Figure from: Alain Le Kouby) (3) Secant column auger (figure from: BAUER video: https://www.youtube.com/watch?v=SLs_UkhU1nY)

The maximum depth is closely related to the mixing tool used. It can reach 10 m with a trencher, up to 18 m with a rotary tool and even deeper with Cutter Soil Mixing (CSM).

These processes use the dry or wet method. In the wet method, a cement grout is injected at low pressure (less than 1 MPa) in the vicinity of the mixing blades. The wet method is suitable for any type of soil regardless of its water status. The dry method, by contrast, is suitable for soils with a high moisture content, able to hydrate the binders injected into the soil in dry form. The main advantage of this process is that very soft soils, including organic soils, can be stabilised at great depth, with high yields, a competitive price and very little excavated material. If the water status is not sufficiently high, the binder will not set with the soil in a uniform manner (formation of lumps). Injection of water at the mixing tool should then be considered. The tool used is thus adapted to either of these processes.

As an example, the dry method is most often used on Loire levees.

Note that working at low outside temperatures (< 0°C) is also possible.

Speed of progress:

With a trencher, it is possible to cut a length in the order of 100 m per day, for depths in the order of 6 to 8 m (i.e., 600 to 800m²/day). This technique offers the best speeds.

The other techniques allow a maximum of 25m^3/h (approximately 300 m^2/day)

Dealing with singular points (ends, enclosed and penetrating structures, etc.):

One day after the production day, the structure must be reworked over a few metres (length to be defined depending on the tool) to avoid discontinuities in the structure. The procedure is to be formalised during the preparatory phase.

Different methods for crossing penetrating structures (pipes and underground networks in particular) have been developed in recent years on Loire levees using a trencher. Information on this subject can be found in the referenced articles.

Site remediation:

Including:

- repair of the dike after the works;
- repair of roadways;
- plant cover of slopes where appropriate;
- removal of storage areas and site roads.

Variants:

They are based exclusively on the nature of the hydraulic binder: cement is generally used to create soil concrete, but for specific needs (associated functions, etc.) it is possible to use other binders such as quicklime, fly ash, etc. The choice is of course dependent on the composition of the ground.

Other construction considerations?

Work site:

- construction of an access road;
- development of the work area;
- creation or development of a storage area.

Two methods can be proposed (example of the trencher):

- wet method:
  - cutting of a pre-trench of defined depth (generally around 1.5 m) by the contractor in the preparation phase;
  - inserting the tool in the soil to the required depth;
  - introduction of binder in the form of grout (with a defined W/C ratio) during the mixing phase;
- dry method:
  - cutting of a pre-trench of defined depth (generally around 1.5 m) by the contractor in the preparation phase;
  - application of the binder in powder form in the pre-trench according to the chosen content;
  - placing the machine above the pre-trench; the blade moves from an initial near-horizontal position to a vertical position with the chain rotating (Figure 3.148);
  - injection of water (this can be done via the machine blade during mixing, for example).

- end-of-site cleaning, repair of the dike and roadway if necessary

Figure 3.148: Machine used to mixed soil trenches using the dry method (figure from: DREAL CVL / DETL)

<u>Required equipment and materials:</u>

Depending on the technique, the equipment and materials used will be different, but always including:

- a filler material for binding with the soil in place;
- a machine for mixing the soil in place (trencher, hydromill trench cutter, auger, etc.);
- a device for injecting the filler material (binder, water, grout, etc.).

These mixing techniques are well suited to soft soils but coarse soils can make mixing difficult. Note that the wet method requires a grout plant.

<u>Disturbance and the environment:</u>

The various mixed soil techniques generate vibrations which are systematically measured if there are neighbouring structures or nearby residents, in order to be able to stop site work if certain pre-defined thresholds are reached (creation of disturbance, reaching non-regulatory thresholds, etc.). Feedback shows that cutting trenches in mixed soil produces low levels of vibration.

One of the advantages of these techniques is mixing the soil in place with a hydraulic binder and minimising the production of excavated material (limited to the excavation of the pre-trench). A certain amount of water and binder is added to the existing, loose soil, but most of it will occupy the space provided by the mixed zone. The surplus (spoil) is put in a skip and then sent to landfill.

<u>Construction checks:</u>

<u>Creation of an inspection zone upstream and downstream of the cut-off wall (example of the trencher):</u>

- this allows the structure to be monitored during site work from fresh samples, for 7, 28, then 56 and/or 90 days tests, or by core drilling for medium- and long-terms monitoring;
- it also allows the machine parameters to be checked when going into the production phase.

The inspection zone should, as far as possible, be isolated from the trencher installation and removal areas.

<u>Checks during the construction phase:</u>

- quantity of hydraulic binder applied (dry method);
- real-time recording of the machine parameters: speed of progress, depth of the blade, amount of water injected (dry method) or grout injected (wet method), chain rotation speed, mixing index, etc.
- laboratory tests at 28 days for permeability and uniaxial compressive strength on fresh samples as recommended in the special technical specifications. For the mixed soil trench technique, a permeability criterion of 10^{-8} m/s can be used as well as a uniaxial compressive strength criterion of between 0.3 and 1.5 MPa at 28 or 90 days.

Figure 3.149 Tool used to take samples at different depths (figure from: DREAL CVL / DETL)

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3.2 3 Technique factsheet: Sheet pile wall (internal erosion)

This technique is a combined contribution by the following authors:

Authors: Edina Koch, Richard Ray (Széchenyi István University), Márton Maller, Gergely Bartal, North-Transdanubian Water Directorate, Hungary.

Authors: Hirotoshi Mori, Shunsuke Sako, Japan

For the corresponding case studies, please refer to Section 3.1.3 & 3.1.4.

Failure mechanism(s) and/or basic function

The primary purpose of the waterside sheet pile wall is to mitigate internal erosion (piping) by increasing the seepage path.

This method is employed to prevent piping by installing a low-permeable wall in the subsoils at the river side toe of river levee, thereby reducing the amount of seepage water and pore pressure that permeates from the river into the subsoils. The technique improves the stability of the subsoil, mitigate the hydraulic failure.

Description technique

The basic structure of the cutoff wall is illustrated in Figure 3.150.

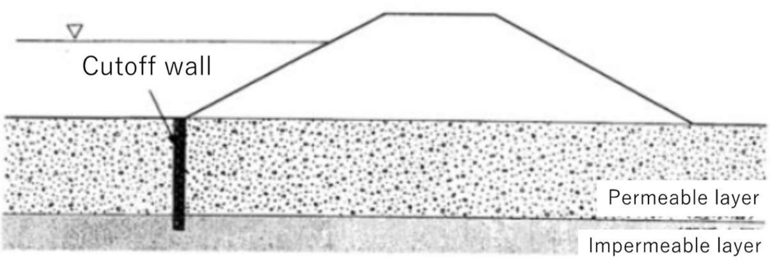

Figure 3.150 Basic structure of a cutoff wall (the Design Guideline for River Levees)

The cutoff wall must meet the piping safety criteria shown in
Table 3.3 (*Japan*) with the results of the seepage analysis. It is important to control the lower water head / pore pressure at the land side toe of the river levee. To achieve the lower water head, the depth to install the cutoff wall is important as deep wall has high performance to reduce water head.

Applicability

Hungary
The sheet pile wall can be installed from the crest, from the landside, or both side of the levee. The subsoil may consist of fine-grained blanket materials, transition soils or sandy-gravelly soils.

Japan
The cutoff wall is a widely utilized countermeasure for internal erosion (piping) in Japan, particularly in subsoils composed of sandy or gravel soil.

This method is applicable when the subsoil is highly permeable. However, if the permeable layer is thick, it is necessary to lengthen the length of the cutoff wall, which may cause economic and construction problems.

Although the sheet pile is regarded as highly effective as cutoff wall, it is dependent on the ground conditions. Gravels or rocks in subsoil might result in the joints may be damaged during construction, resulting in a loss of the water sealing effect. Therefore, it is essential to employ appropriate construction methods. Additionally, both the sheet pile method and the continuous underground wall method necessitate a specific area for construction and a designated route for heavy machinery and equipment. Notably, the latter method involves the use of larger construction machinery, which requires a larger space than the sheet pile method. It is also necessary to consider the bearing capacity of the ground surface.

Design

Design criteria

Hungary

The design primarily follows an Ultimate Limit State (ULS) approach, specifically focusing on verification against hydraulic failure (HYD) and overall stability (GEO) in accordance with Eurocode 7. Design Approach 3 (DA3) is specified by the Hungarian National Annex for overall stability (GEO), while DA2* is applied to other limit states, including hydraulic failure (HYD). The GEO Ultimate Limit State (overall stability) refers to the failure or excessive deformation of the ground, where the strength of the soil provides the significant resistance. In accordance with DA3, partial factors are applied to actions and ground strength parameters simultaneously. For the Hydraulic Limit State (HYD), the calculations account for the design destabilizing total pore water pressure (or seepage force) versus the design stabilizing total vertical stress (or submerged weight) of the levee and its foundation, applying the partial safety factors prescribed by the standard.

For characteristic water levels on the waterside, the standard flood level (MÁSZ) given in Hungarian governmental regulations serves as an estimated maximum stage following EC7-1 Section 2.4.5.3.(1)P. The level used in the design is typically MÁSZ+1.0m. Analyses have shown that this level corresponds to a return period of about 1000 years.

Japan

The cutoff wall must meet the piping safety criteria shown in
Table 3.3 with the results of the seepage analysis. The Japanese Design Guideline for River Levees (2012) by the Japan Institute of Country-ology and Engineering (JICE) contains the specifications of the design criteria.

Which factors affect the dimensioning?

The main design parameter for underseepage is hydraulic conductivity. The soil stratigraphy should be determined accurately.

Generally, sheet pile walls can be an option to increase the overall stability of the levee. In that case, the primary parameter is the shear strength of the soils.

Proper design requires numerical modeling to assess the interactions of soil strength, seepage forces, and effective confinement conditions in the levee. Such modeling requires expert knowledge to produce accurate results.

How are key parameters for this technique measured or estimated?

Conventional site investigation methods, e.g. boreholes, CPTu soundings, geophysical measurements, and laboratory tests can be used to determine the input parameters for both analytical and numerical models..

Other design considerations

Other design considerations are related to:

- increasing flood frequency, intensity, or duration due to climate change;
- land subsidence;
- environmental considerations;

The cutoff wall construction may cause so-called groundwater obstructions such as wetting of the land within the embankment or depletion of wells, depending on the ground and groundwater conditions, so sufficient attention should be paid to this point as well.

Construction

Which methods of construction are available?

There are different solutions for the execution of the sheet pile wall. The installation can be vibration, driven or pushing.

Hungary
In the Szigetköz floodplain area, vibration was used due to the thin blanket layer and thick gravel layer.

The technique consists of the following construction phases:
1) layout of the wall;
2) removal of the humus;
3) excavation of the head trench;
4) vibration of the sheet piles;
5) fill in the trench with the excavated soil and humus;
5) grassing.

Japan
Figure 3.151 shows the construction and its machinery for steel sheet pile injection. Some construction machines use water jet to insert sheet piles to hard subsoil. Construction equipment manufacturers research and develop various products and supply them to the market.

Figure 3.151 Left: Construction of sheet piles, Right: Construction machinery for a sheet pile

Other construction considerations?

The installation technology depends on the subsoil.
The installed sheet pile wall forms a natural obstacle in the ground.
Noise or vibration caused by construction, especially installation of the sheet piles, may affect nearby residents or animals.

Variants on the technique or construction method

Common practice uses U or Z steel profiles. But the material of the sheet elements can be different e.g. fiberglass sheet pile, vinyl sheet pile.

Alternative cut-off walls and construction methods in Japan

Cutoff wall can be classified into sheet pile, continuous underground wall and grouting. The characteristics of each are summarized in Table 3.11. Sheet pile method: Steel sheet pile and continuous underground wall method: slurry trench are major cutoff wall types in Japan.

Table 3.11 Cutoff wall types and features (the Design Guideline for River Levees)

Types	Materials	Features
Sheet pile method	Steel sheet pile	· Used widely due to its superior workability. · There is leakage through the joints, especially when gravel soil is targeted, which can cause the joints to open up and reduce the applicability by half.
	Concrete sheet pile	· RC sheet piles and PC sheet piles are available. · The length is limited to 5 m or less, and there are few examples of its use.
	Thin steel sheet pile watertight wall	· Wide, thin steel sheets are cast using a combination of vibrohammer and water jet, and the joints are filled with grout material to ensure watertightness. · Compared to steel sheet piles, this method is more economical and watertight, but its thinness prevents forced penetration, and its workability may be a problem in some ground conditions.
Continuous underground wall method	Slurry trench	· A trench is excavated into the ground and backfilled with a mixture of excavated soil, bentonite, and cement to create an impervious wall. · A method has also been developed in which a soft vinyl chloride sheet is inserted into the slurry trench as a watertight material to improve watertightness.

		• Overseas, this method has been used as a watertight barrier for fill dams and river levees. Although there are few examples in Japan, this method is economical, easy to construct, and can be applied to a wide variety of ground types.
	Concrete wall	• This method uses trenches to construct concrete walls, but has had little success in rivers due to its economic efficiency and ease of construction.
Grouting method	Cement grouting	• Cement milk or waterproofing chemicals are injected into the foundation ground, which is easy to install, but the applicability and durability of waterproofing are unknown.
	Chemical injection	

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3.2.4 Technique factsheet: Vertical filtration & drainage in the foundation (internal erosion)

Main authors: Gregor Portmann & Hansjörg Vogt (Tragweite AG, LI)
Contributors: Florin Banzer (Sprenger & Steiner AG, LI); Johannes Toepke (IUB Engineering AG, CH), Anne Christine Pfaffen (Authority for Civil Protection ABS, Principality of Liechtenstein)

For the corresponding case study, please refer to Section 3.1.5.

Failure mechanism(s) and/or Basic function

Bottom heave; Slope sliding; Internal erosion.
The filter gravel column serves to reduce pore water pressure beneath the flood deposits in the event of flooding, thereby improving levee stability on the landside.

Description technique

The overall system for the levee restoration is shown in Figure 3.30 b. Here, the focus lies on the filter gravel columns. The construction process is described below, pictures of the construction process are shown Figure 3.152.

a) The outer casing pipe (Ø 900 mm) is driven into the ground using a vibrator.
b) The pipe is cleared out using a clamshell bucket. The excavated material is documented so that the location of the fine-grained silty flood deposits is known.
c) View of the excavated pipe from above with Rhine gravel at the bottom.
d) Cross-section of the excavated pipe with the inner pipe (Ø 600 mm) already installed. The length of the inner pipe depends on the location of the fine-grained silty flood deposits – the inner pipe must extend approximately 75 cm below this layer. Spacer holders made of reinforcing steel are welded to the inner pipe (4 pieces at the lower end) to ensure a uniform gap between the two pipes.
e) Top view of the two pipes before pouring in the filter gravel.
f) Backfilling with filter gravel: First, the inner pipe is filled, as it does not require a two-part filter structure in the lower section. Only then the outer pipe is filled with the finer filter gravel.
g) Finally, both pipes are simultaneously removed using a vibrator. As the filter gravel compacts during this step, more filter gravel must be added depending on the desired elevation of the pile head. The two-part filter structure is clearly visible.

Figure 3.152: Construction of filter gravel columns: a) Vibrating the outer pipe into place; b) & c) Excavation with clamshell bucket; d) & e) Installation of the inner pipe; f) Backfilling with filter gravel; g) Finished filter gravel column.

Finally, Figure 3.153 shows the construction of the cantilever retaining wall as part of the dam remediation work. Figure 3.153a shows a front view of the wall, with the outlet openings visible. The filter gravel is covered with a geotextile, and some of the ballast fill has been installed. Figure 3.153b shows a similar situation, but here the ballast fill has not yet been installed. The drainage openings in the base plate and the drainage pipe running through the filter gravel are clearly visible. This pipe is intended to prevent seepage water leaking through the wall outlet in the event of frequent flooding (HQ30, high discharge with an annual probability of occurrence of 3 percent), as part of the serviceability measures.

Figure 3.153: Dam remediation during construction progress: a) Front view of the cantilever retaining wall with outlet openings; b) Side view of the cantilever retaining wall with drainage openings in the base plate.

Applicability

The filter gravel columns pressure relief system presented here is suitable for any levee system in which pore water pressures below an impermeable interlayer have a negative impact on levee stability. It is particularly suitable for situations where space is limited and no pressure relief trench can be constructed at the foot of the levee (see Figure 3.30).

Design

Design criteria

The design is primarily an ultimate limit state approach. The levee construction has to perform during peak hydraulic loads. Serviceability may become important if the amount of seepage water is to be limited. Due to the restoration measures, this will be greater than before the construction work.

Which factors affect the dimensioning?

A distinction is made here between the design of the filter gravel as a material and the design of the filter gravel column as such.

Filter gravel: Hydraulic efficiency and mechanical stability are particularly important factors in the design of the filter gravel. Suffusion is a particularly important issue here, especially for gap-graded materials. A good overview is provided in the design aid Material Transport in Soil (Bundesanstalt für Wasserbau, 2013). Put simply, the aim is to produce filter gravel that is as coarse as possible (permeability) and as fine as necessary (stability).

Filter gravel columns: The dimensioning of filter gravel columns is a complex spatial problem and depends on a number of influencing factors (permeability of the materials, diameter, embedment depth & filter length, spacing between the individual columns, hydraulic gradient, etc.) and is usually solved with the aid of stationary FE seepage analyses. It is particularly important to ensure that the distance between the individual columns is not too large, as otherwise they might lose their effectiveness in the intermediate areas (Brandl & Szabo, 2015).

How are key parameters for this technique Measured or estimated?

The soil parameters are determined with conventional methods.

In the **field**, exploratory trenches are usually excavated, the material encountered is assessed by geologists and samples are taken for further investigation. Core drilling and dynamic penetration tests (location of flood deposits) are also carried out.

In the **laboratory**, sieve curves are usually produced and, depending on requirements, tests are carried out to determine shear strength, permeability, optimum water content for compaction, etc.

Other design considerations

In principle, it would have been possible to omit the two-stage filter structure if either a type of casing pipe or a type of geotextile could be permanently installed in the area of the fine-grained flood deposits. However, both options were rejected for sustainability and durability reasons. The authors particularly believe that installing a geotextile bag to separate the flood deposits and the filter gravel would be almost impossible to control using the construction method presented here, as there is no guarantee that the geotextile would stay in the intended position and remain intact when the piping is pulled.

Construction

Which methods of construction are available?

In principle, all methods for creating cased boreholes can be considered. However, it should be noted here that vibrating the pipes during withdrawal has proven to be beneficial, as this compacts the filter gravel in the pipe.

Other construction considerations?

When vibrating the piping into place, care must be taken not to apply too much energy to the ground, as this can have a negative effect on the levee structure and the surrounding area (settlement, cracks, etc.). In the presented remediation project, for example, the piping was often not vibrated into the ground in one continuous movement, as the resistance was too high without intermediate excavation steps.

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3.2 5 Technique factsheet: Xanthan gum biopolymer-based soil treatment (internal erosion)

Authors: Ilhan Chang (Korea)

For the corresponding case study, please refer to Section 3.1.7.

Failure mechanism(s) and/or Basic function

This study focused on the potential application of Xanthan Gum (XG) Biopolymer for controlling internal erosion in earthen embankments. A full-scale embankment was constructed, and internal erosion was simulated beneath a box culvert to evaluate the effectiveness of XG in mitigating erosion.

The experimental results demonstrated that XG effectively minimized the progression of internal erosion by promoting particle bonding, enhancing apparent cohesion, and blocking soil pores. The XG-treated levee maintained its structural integrity without significant erosion for up to 2,500 seconds.

Figure 3.154 Field-scale application of Xanthan Gum-based soil treatment

Description technique

Chemical Characteristics and Applications of XG

Xanthan Gum (XG) is a polysaccharide biopolymer produced by *Xanthomonas campestris*, consisting of two glucose units, two mannose units, and one glucuronic acid unit, which predominantly forms a helical structure. The conformation of XG solutions may vary between helical and random-coil structures depending on the dissolution temperature and salinity. The viscosity of XG solutions increases linearly with XG content, and exhibits high stability across a wide range of temperatures, pH values, and electrolyte concentrations.

Due to its thermal stability, compatibility with food ingredients, and pseudoplastic flow properties, XG has been widely used in the food industry. In addition, XG has been applied in the petroleum industry as a drilling mud thickener for viscosity control, as well as a gelling and suspending (flocculating) agent. More recently, numerous studies have reported the implementation of XG in geotechnical engineering practice owing to its high soil improvement efficiency and reasonable economic feasibility.

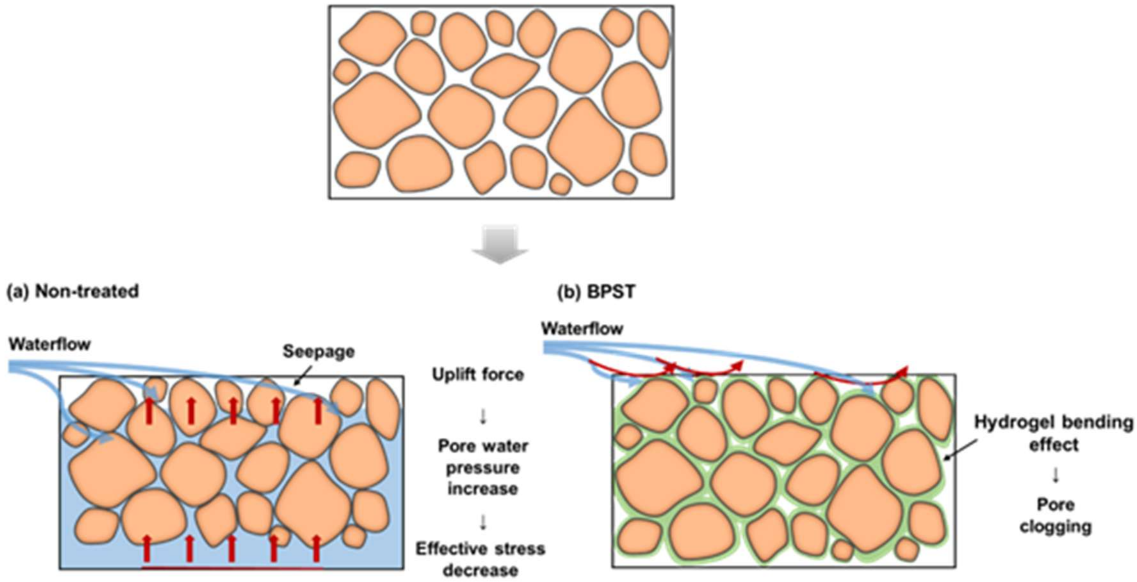

Figure 3.155 Mechanisms of Soil Improvement by Biopolymers

Effect of XG BPST on Mitigation of Internal Erosion in Soil

As illustrated in Figure 3.155(a), when water flow occurs in an untreated earthen levee, seepage propagates through the soil pores, and buoyant forces develop as the water level rises. Consequently, pore water pressure increases while effective stress decreases, resulting in a reduction of the levee material strength.

In contrast, when water flow occurs in a Biopolymer-treated earthen levee (Figure 3.155(b)), the bending effect of the Biopolymer hydrogel induces the formation of inter-particle bridges and clogs the soil pores. This improvement in permeability characteristics inhibits sudden infiltration and thereby contributes to maintaining the stability of the levee.

Applicability

This technique can be applied to earth dam structures. At the study site, the subsurface geology consisted of a surficial alluvial soil layer (SPT N = 6–28, within approximately 8–9 m), underlain by weathered soils and bedrock (SPT N > 44). Such stratigraphic conditions are commonly observed in actual river levees and rural levees.

In this study, a full-scale levee with a height of 3 m, width of 10 m, length of 15 m, and side slope ratio of 2H:1V was constructed to verify the effectiveness of XG BPST in mitigating internal erosion. It is particularly noteworthy that levees are often constructed in conjunction with hydraulic structures such as culverts. Interfaces between these structures and the earth fill material frequently become unfavorable seepage pathways where interface erosion is prone to occur. This technique has demonstrated the potential to suppress internal erosion even in these vulnerable sections.

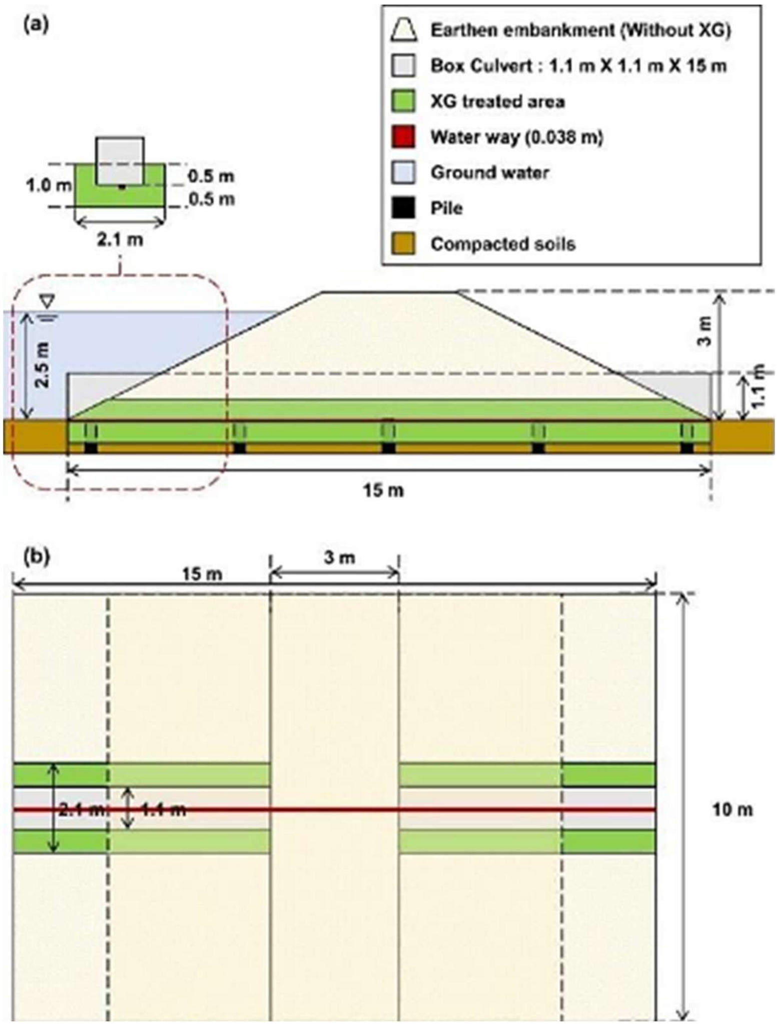

Figure 3.156 Design Cross-Section and Plan View of the Levee

Therefore, XG BPST can be applied as an eco-friendly reinforcement technique to prevent internal erosion and to ensure long-term stability in various earth dam structures, including river levee.

Design

Design criteria

The design criteria for the levee construction adopted in this study classify the resistance of geotechnical materials to piping and cracking based on particle size distribution and Atterberg limits. According to the design criteria, standard test procedures such as particle size analysis (ASTM D6913), fall cone test (BS EN ISO 17892), and rolling thread test (ASTM D4318) were employed to classify soils with different compositions and XG contents.

Table 3.12 Design criteria for classifying the piping and cracking resistance of soils used in levee construction.

Category	Grade	Material	Characteristics
Piping resistance	1	CL, CH with PI > 15, Well graded SC with PI > 15	Greatest resistance to piping.
	2	CL, ML with PI < 15, Well graded SC and GC with PI = 7–15	Intermediated resistance to piping.
	3	SP, Uniform SM, ML with PI < 7	Least resistance to piping.
Cracking resistance	A	CH with D_{50} < 0.02 mm and PI > 20	Least chance for cracking.
	B	C, SC, SM, SP with D_{50} > 0.15 mm	Little chance for cracking.
	C	CL, ML, SM with PI < 20, D_{50} = 0.15–0.02 mm	Vulnerable to cracking.

Dimensioning

In determining the dimensions of a levee, the primary factors include the resistance to piping and cracking based on soil gradation and Atterberg limits, compaction characteristics and water content, unconfined compressive strength, and economic feasibility. For the design process, empirical classification criteria, laboratory testing, and conceptual as well as numerical analysis models may be employed.

Key parameters

The key parameters of this technique were measured through standard tests, including particle size analysis (ASTM D6913), Atterberg limits (ASTM D4318), compaction test (ASTM D698), and unconfined compressive strength (UCS, ASTM D2166).

Other design considerations

The durability limitations of the Biopolymer (XG) are an important consideration for long-term applications. To address this issue, chemical modifications such as XG–Cr^{3+} composite treatment have been investigated, which enhance cross-linking bonds and thereby improve resistance to prolonged seepage flows and environmental changes, including repeated cycles of flooding and dry periods.

Construction

Which methods of construction are available?

In this study, dry soil and XG powder were thoroughly mixed using a backhoe to produce a uniform XG-treated soil. A box culvert was installed, and a levee was constructed using soil treated with 1% XG. The box culvert was placed on piles.

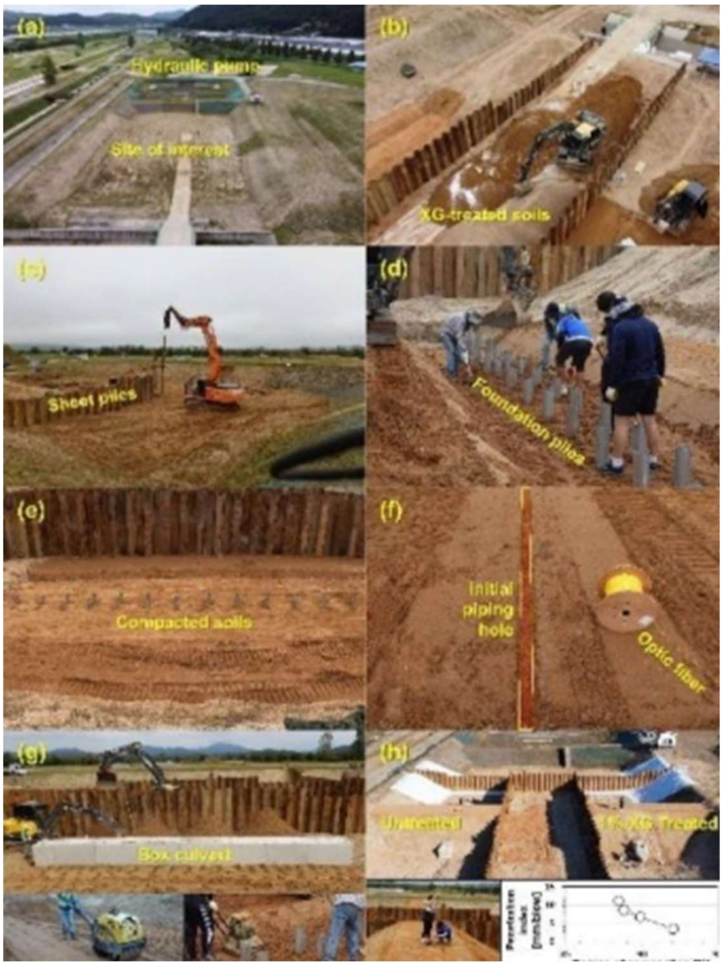

Figure 3.157: Construction process of the full-scale levee

Beyond this study, several applications of XG treatment for levee reinforcement have been conducted. One example is the BPST double-channel nozzle spraying method implemented in Andong, Republic of Korea. In this method, the biopolymer solution and in-situ soil are separately pressurized through a hydraulic pump and a pneumatic pump, respectively, and then sprayed onto the levee slope via a dual-channel nozzle. In the present study, the biopolymer solution and soil mixed on site were sprayed through a dual-channel nozzle located at the end of a separate pipe, thereby forming a BPST layer of approximately 15–20 cm thickness on the levee surface.

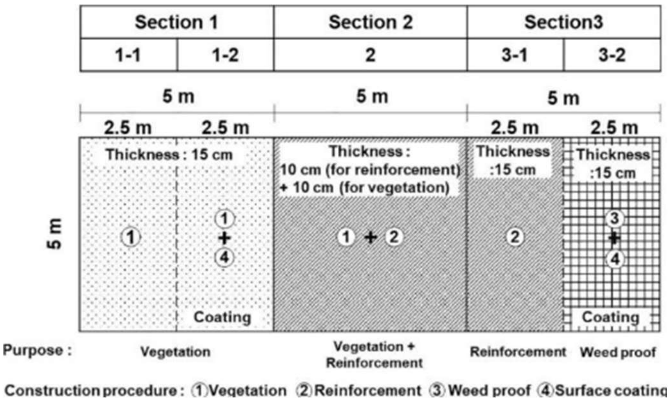

Figure 3.158 Construction plan of BPST implementation for slope surface protection

The second method is the wet-spray application. In this method, all components are premixed in a mixing tank and then sprayed over a wide area through a nozzle using an air compressor. Because the components are thoroughly mixed prior to spraying, this method provides improved quality control. In this study as well, in-situ soil was mixed with the biopolymer compound and sprayed onto the slope surface.

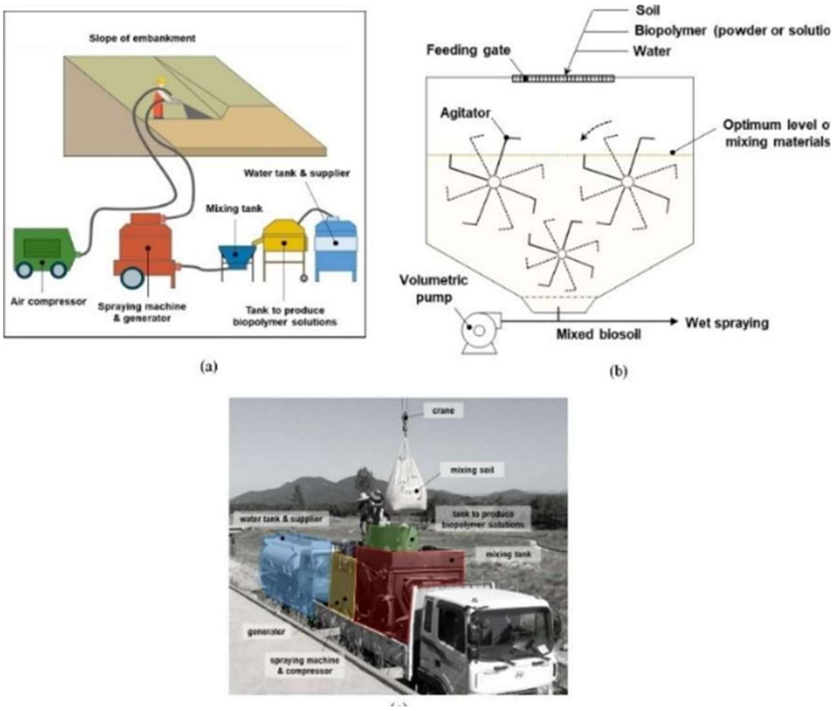

Figure 3.159 In-situ wet-spraying system used in this study. (a) System configuration.

Other construction considerations?

For future studies, laboratory-scale experiments, including pore erosion tests and numerical analyses, are required for more comprehensive validation. Through such efforts, the effects of XG treatment can be systematically evaluated, and the correlation between pipe failure and the location and dimensions of pipe voids can be identified.

Variants on the technique or construction method

The mixing ratio and application method of the Biopolymer must be adjusted according to the soil type and properties at the site. For example, the optimum dosage may differ between sandy soils and fine-grained soils. Therefore, selecting the appropriate proportion and construction method suited to the characteristics of the levee material is essential to maximizing the effectiveness of internal erosion mitigation.

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Chang, I., Im, J., Prasidhi, A. K., & Cho, G. C. (2015). Effects of Xanthan gum biopolymer on soil strengthening. *Construction and Building Materials*, *74*, 65-72.

Chang, I., Lee, M., Tran, A. T. P., Lee, S., Kwon, Y. M., Im, J., & Cho, G. C. (2020). Review on biopolymer-based soil treatment (BPST) technology in geotechnical engineering practices. *Transportation Geotechnics*, *24*, 100385.

Seo, S., Lee, M., Im, J., Kwon, Y. M., Chung, M. K., Cho, G. C., & Chang, I. (2021). Site application of biopolymer-based soil treatment (BPST) for slope surface protection: In-situ wet-spraying method and strengthening effect verification. *Construction and Building Materials*, *307*, 124983.

SIMSG ISSMGE

3.2 6 *Technique factsheet: Geosynthetic honeycomb cells filled with concrete (external erosion)*

Author: José A. Walters Monteiro, TECNICA Engenheiros Consultores, Lda, Mozambique.

For the corresponding case study, please refer to Section 3.1.9.

Failure mechanism(s) and/or Basic function

Primary: External erosion on the water side. Secondary: land side erosion and internal erosion at defects.
Basic function: Provide an armour layer to resist hydraulic actions and prevent soil loss through a filter system.

Description technique

Armour layer of geosynthetic honeycomb cells filled with concrete, anchored into the toe and complemented by gabions. Figures in the Remedial measures paragraph illustrate the layout, cross-section and discharge cascade.

Applicability

Type of setting: river levees in rural areas. Subsurface: sandy alluvium with clay and silt layers. Applicable to levees without embedded structures, with local concrete production available.

Design

Design criteria

Design criteria included global stability of slope by Bishop method (dry, saturated and rapid drawdown cases). The armour layer of geosynthetic honeycomb cells filled with concrete solution was based in previous experience in roads and embankment protection in the Limpopo River levees after the 2000 and 2013 floods.

Which factors affect the dimensioning?

Hydraulic loading: design water levels and velocities derived from flood-frequency analysis and hydraulic modelling.

Material parameters such as the unit weight and friction angle impact the stability of the structure.

How are key parameters for this technique measured or estimated?

Topography/bathymetry for riverbed and land cross-sections; orange displacement method for water velocity measurements, pits and laboratory tests for foundation soil classification, permeability and strength; catalogues for geotextile properties; concrete mix design.
Observed flood marks, interview of residents and satellite imagery to validate design assumptions post-event.

Other design considerations

Considerations included:

- Potential increase in flood frequency/intensity/duration under climate change; allowance for freeboard and toe robustness.
- Dry seasons and potential desiccation cracking in earthen faces (mitigated by armour and filter).
- Material availability and costs; environmental footprint; population growth in flood-prone areas; constructability windows.

Construction

Which methods of construction are available?

Methods: strip and trim slope; foundation preparation; install geotextile with adequate overlaps; placement of geosynthetic honeycomb cells filled with concrete; finishing of concrete works; construct toe trench; gabion works, and construction of complementary unlined drainage channel.

Equipment and crews: light earthmoving equipment and manual placement suitable for rural settings.

Quality control: topographic control of settings, slopes and levels, visual and checklist-based acceptance (finishing), compaction tests, concrete strength certificates.

Figure 3.160 Construction of gabions wing Wall, river side

Figure 3.161 Execution of toe protection on the river side

Figure 3.162 Execution of spillway on the river side.

Figure 3.163 Execution of the spillway crest (access for maintenance)

Figure 3.164 The spillway crest and works on the cascade, land side

Figure 3.165 Execution of the cascade

Figure 3.166 Toe protection channel on the land side

Other construction considerations?

Community access constraints required safety measures near the works. Construction was performed during the dry season making it more simple to build the anchor gabion toe.

Variants on the technique or construction method

Variants may include gabion toe-only protection with vegetated slope cover or armour layer made of articulated concrete blocks with geotextile and filter.

References

Design manuals/standards/codes of practice

Detailed Design Project Report for Emergency Rehabilitation Works of Nante Levees (2016). TECNICA Lda.

The International Levee Hand Book, US Army Corps of Engineers, CIRIA, Ministérie de lÉcologie de Développement Durable et de lÉnnergie; CIRIA,2013

DNGRH/ARA Centro-Norte hydrometric records.

Technical Standard and Guidelines for Design of Flood Control Structures, Department of Public Works and Highways & Japan International Cooperation Agency (JICA), June 2010

Bank Protection Rivers and Lochs, SEPA – Scotish Environment Protection Agency, April 2008

SATCC Standard Specifications for the Roads and Bridge Works, September 1990, Reprinted July 2001

Publications

Dutch Risk Reduction Team – Reducing the Risk of Water Related Disasters; DRR – Team Mission Report – Mozambique – Licungo Basin – June 2015

Estimation of flood-exposed population in data-scarce regions combining satellite imagery and high resolution hydrological-hydraulic modelling: A case study in the Licungo basin (Mozambique). Luiz Cea At all. Journal of Hydrology: Regional Studies 44 (2022) 101247

3.2 7 Technique factsheet: Shortcrete retaining wall (external erosion)

This contribution is based on: FICHE TECHNIQUE Protection externe par la reprise d'un mur par béton projeté. External protection thanks to the recovery of a wall by shotcrete. From Recueil de méthodes et de techniques de confortement et réparation des digues de protection en remblai of the Comité Français des Barrages et Reservoirs (CFBR).

Main authors: T. MONIER (Artélia)
Contributors: M. SUTTER (INRAE); D. POULAIN (INRAE)
Reviewers: PL. REGAZZONI; S. PATOUILLARD (DREAL Centre-Val de Loire); B. CHALUS (CNR)
Final Approval: A. RULLIERE (INRAE)

Failure mechanism(s) and/or Basic function

Primary Functions

The primary function sought is to resist external erosion on the upstream side (erosion by water current, wave action, resistance to swell) and to protect and extend the life of the body of the levee. This type of consolidation also generally fulfils a retaining wall function by way of anchorings and a wall of particular size (thickness) to achieve this objective. These anchorings also enable the shotcrete reinforcement to play a containment role by preventing loss of materials from the reinforced wall.

Secondary Functions

Spayed concrete shells can also provide an imperviousness function due to the characteristics of the materials they consist of.

Description technique

The term "wall" as used in this sheet refers to a structure with a vertical or quasi-vertical facing generally constructed as a retaining wall or for protection purposes.

The sprayed concrete technique consists of applying a layer of concrete under pressure onto a reinforcement mesh.

Reinforced concrete walls and embedded stone pitching are components of flood defence systems. Often old, they sometimes exhibit very significant degradation that a conventional maintenance operation alone will not be able to correct: high erosion, cracking and differential subsidence.

Repair of these works by complete or partial structural renovation of their structure with a shotcrete shell (Figure 3.167) enables such corrections to be made while limiting immediate environmental disturbance and optimising the consolidation work (economically and in the amount of time required).

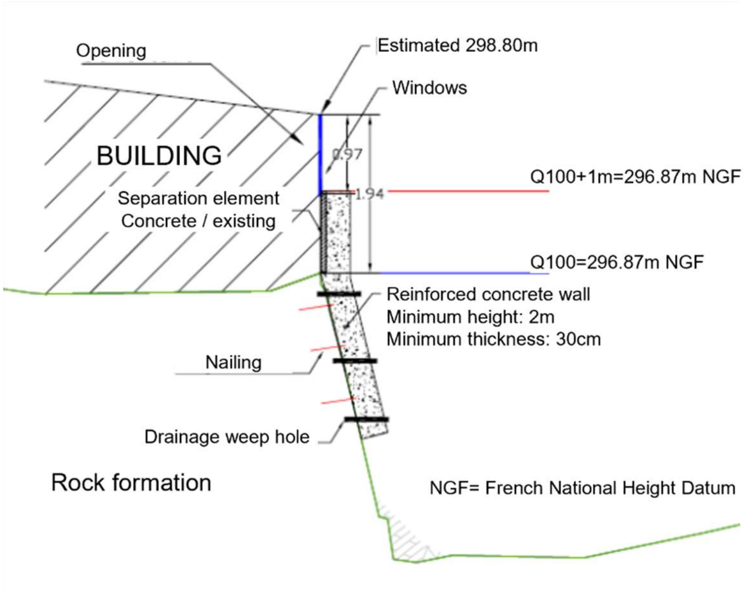

Figure 3.167 Example of wall renovation using sprayed concrete (Source: ARTELIA)

Applicability

For new works and for consolidation work, this technique is commonly used in all types of construction: underground structures, road embankments, coastal and river works and so on.

The technique is generally used on the upper part of banks (zone out of water at least occasionally). In these cases, it is therefore not designed to take account of the risk of toe scour that may occur when the bed of the watercourse consists of material that can move. It is therefore generally necessary to add other techniques to this type of reinforcement to address this risk (see the sheets relating to external protection).

Design

The expected performance of a sprayed concrete reinforcement is designed to fulfil its design functions. Therefore, in terms of external protection, the consolidation will have to withstand different external forces from the action of waves, flows, floating objects and chemical attack from the environment.

In terms of its retaining wall function, the anchorings and sprayed concrete shell (concrete and rebar) will be required to withstand tensile and flexural forces due to earth pressure, water pressure and possible external overloads:

- Action due to static water pressure.
- Water action under dynamic conditions (current forces).
- Internal forces to be taken up (during flooding and fall in water levels).

Design criteria

A shotcrete reinforcement can be designed by assuming that the reinforced structure will have monolithic behaviour and by applying the Eurocodes: NF EN 1990: Eurocode 0 - Basis for calculation of structures; - NF IN 1991: Eurocode 1 - Actions on Structures; - NF IN 1992: Eurocode 2 - Calculation of reinforced concrete structures.

For a structure, these standards consider different design aspects by referring to:

- Calculations of the theoretical deflection;
- Calculations of the maximum shear force and maximum bending moment.

In addition to this structural design and verification, the constituent materials of the structure must be designed and the adjoining components to the structure must be defined.

- Design of sprayed (reinforced) shotcrete;
- Design of rock anchorings (length of restraint of rebar);
- Design of drainage systems if necessary (aspect not covered in this sheet).

In addition, the overall stability of the reinforced structure must be checked.

Which factors affect the dimensioning?

The data required for the design and sizing of the geometry of the structure are in particular:

- For general characteristics: weather and climatic characteristics of the site (notably frost, high temperatures, but also humidity).
- In a marine environment, marine and coastal hydraulic characterisation of wind data, water levels, tide, surges, swell, rise in sea level, design basis water level;
- In a river environment, hydraulic characterisation of the river with respect to characteristic water heights, current velocity, turbulence, wave action;
- For geotechnical data:
- o Nature of materials in place and their geometric distribution;
- o Mechanical stability of materials in place and their ability to take up loads, including in the event of the toe of the wall being eroded, particularly taking into account the nature of the connection between the wall and foundation;
- o The aggressiveness of the water on site (distinction between marine and fresh water environments), which is particularly important for the life of the work and is reflected in terms of design in the choice of concrete, reinforcement and thickness of concrete cover.

This data is used to determine the expected characteristics of the concrete and more generally of the consolidation measures to be implemented.

How are key parameters for this technique Measured or estimated?

Choice of concrete

The specifications of the sprayed concrete are defined in the standards for sprayed concrete: NF EN 14 487-1 and NF EN 14 487-2. These relate in particular to:

- o Granularity: 0 to 8 mm (AFTES range of sizes);
- o Cement density: 1850 kg/m^3;

- Compression strength at 28 days: > 30 N/mm² for C 30/37; > 35 N/mm² for C 35/45; > 40 N/mm² for C 40/50.

Depending on the situation, the following criteria are added to the general criteria:
- Nature of the admixtures (setting agent etc.);
- Origin of aggregates;
- Limiting losses by rebound and the difference between concrete in place and the initial mix.
- The nature and quality of reinforcements (fibres if relevant);
- Quality of concrete cover on the reinforcement;
- Sustainability criteria (as defined in Case Study 2).

Strength Calculation of concrete casing
Specific calculations of flood and receding water strength involve:

- Static thrust calculation:

$$\sigma(kPa) = H * \gamma_w$$

Where:
γ_w is the density of water in kN/m^3.
H is the water height in m, the difference between the free surface of the groundwater and the level at which the thrust pressure is calculated.

- Calculation of current action:

According to Eurocode 0 part 4.9, the intensity of the total horizontal force Fwa (N) exerted by the currents on the vertical surface should be determined by the following expression:

$$F_{wa} = \frac{1}{2} k \rho_{wa} h b v_{wa}^2$$

Where:
v_{wa} is the average water velocity in relation to its depth, expressed in m/s;
ρ_{wa} is the density of water, expressed in kg/m^3;
h is the depth of the water, excluding the depth of local scouring, expressed in m;
b is the width of the obstacle, expressed in m;
k is the form coefficient where:
  - k = 1.44 for an obstacle of square or rectangular cross-section perpendicular to the current;
  - k = 0.70 for an obstacle of circular cross-section in the plane.

The results of these preliminary calculations constitute the actions to be taken into account during flooding and fall in water level when calculating the internal forces to be taken up.

Study of final retaining wall (Standard NF EN 1992-1-1),
The following calculations are usually design basis, however it may be necessary to adapt them for a particular configuration:
- During Flooding
  - Calculation of the internal forces to be taken up (bending moment and maximum shear force) by the structure;
  - Design of reinforced sprayed concrete;
  - Calculations of the theoretical deflection;
- During fall in water level
  - Design of rock anchorings;

- Calculation of acting forces at ELU (bending moment and maximum shear force);
- Design of reinforced sprayed concrete for fall in water level;
- Design of length of restraint of anchors;

Other design conciderations

In addition to factors common to any consolidation (see introductory chapter), it is necessary to pay attention to the following points:

- Design of
  the connection between the wall and its foundation (paying particular attention in the presence of a scour protection toe);
- Drainage to avoid loading of the facing;
- Access to the site during the work (delivery of ready-mix concrete);
- Foreseeable implementation difficulties (temperature, wind, flood);
- Environmental conditions;
- Maintenance and repair conditions;
- Associated techniques already existing on the structure or implemented as part of the work: temporary cofferdam, renovation of existing structures, crossing structures or transition structures, waste and environmental management.

Construction

Figure 3.168 Situation before Consolidation - La Romanche in St Pierre de Mésage, Isère (Photo credit: ARTELIA)

Figure 3.169 Example of dry spraying - La Romanche in St Pierre de Mésage, Isère (Photo credit: ARTELIA)

Figure 3.170 Example after repairing connection with masonry wall - La Romanche in St Pierre de Mésage, Isère (Photo credit: ARTELIA)

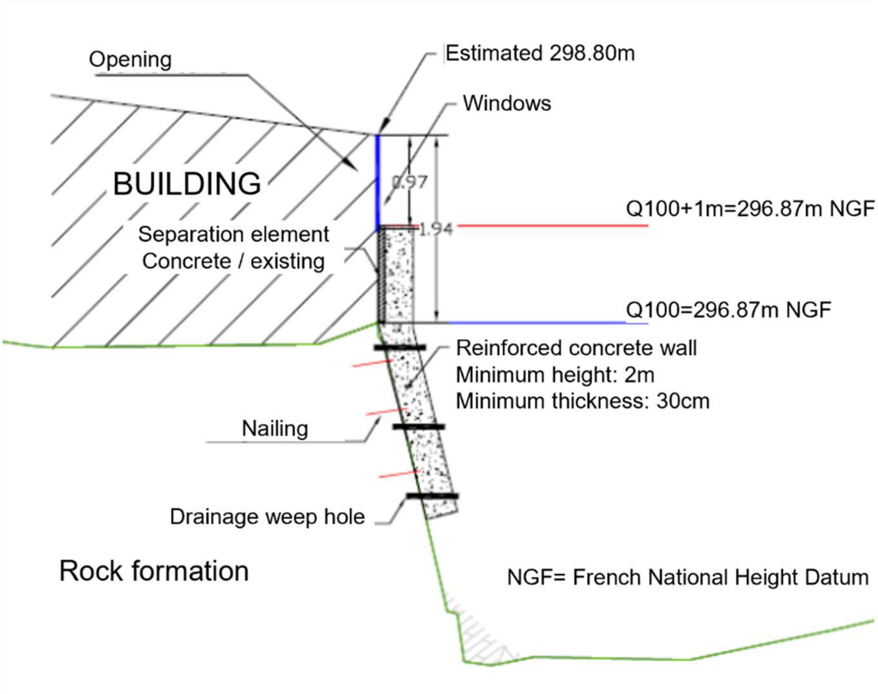

Figure 3.171 Diagram of reinforcement of a sprayed concrete consolidation (Source: ARTELIA)

Figure 3.172 Detail of connection of wall to foundation (Photo credit: ARTELIA)

Which methods of construction are available?

The supply is specified in the Specific Technical Clauses by particularly listing the expected characteristics (see description above).

It will also be necessary to detail constraints in terms of: supply and site conditions, spraying technology (dry or wet), environmental friendliness and so on.

Worksite Execution

Construction of the reinforcement takes place in several steps:

- Surveying and treatment of the existing facing; a clean, sound and unfrozen substrate is required. This may require treatment before concrete is applied: cleaning and re-pointing of masonry - renovation of expansion joints on the existing wall using a mastic resin;
- Installation of one or more layers of reinforcing mesh anchored to the substrate with interposed weep holes (for drainage of the ground behind the facing);
- Application of concrete by spraying. Usually this is done as follows: the concrete is propelled by compressed air in pipes to the spray nozzle. Water arrives at the nozzle separately and is metered there to wet the mixture in the desired proportion and consistency (generally S3 - very plastic concrete with Abrams cone slum control), just at the moment of the spraying onto the wall.

We note a few particularities to be taken into account when carrying out shotcrete projects:

- Air and substrate temperature range for application: + 5° to + 35° C (depending on the specifications associated with each type of concrete);
- Application in layers of 20 to 40 mm for a total cover thickness of between 80 and 200 mm.

Materials and Equipment Required

- Pressure jet cleaning equipment;
- Specific materials associated with the implemented component: concrete, steel reinforcement and anchorings;
- Equipment for inspecting performance of the work on site (sampling, cone);

- Conventional spraying equipment: rotor or peristaltic machine, spraying silo, spray nozzle, storage hopper (Figure 3.173);

Figure 3.173 Shotcrete work site (Photo credit: ARTELIA)

- Equipment requiring specific skills: admixture metering device, robot, water suppressor, skilled labour (nozzle orientation and control, day joints);
- Weep holes (density generally of the order of 1 per 4 m²), rendered and perforated in the interior and equipped with a water permeable non-biodegradable geotextile, which obstructs all solid materials.

Other construction considerations

Disruption/ Environment

Environmental points to be watched include: noise, dust raised by site machinery, particulate pollution from fines (in water), accidental pollution, site waste management, wildlife protection.

Performance checks (including suitability tests)

- Approval of the concrete formulation;
- Approval of the aggregates used;
- Validation of Quality Assurance Procedures and construction work procedures.
- Supply control, consistency check by slump-test (Abrams cone: NF EN 12350-2 - Tests for fresh concrete - Part 2: Slump Test);
- Inspection of reinforcement (ST type reinforcing mesh, HA type anchoring, etc.), verification of compliance with execution plan (position, diameter, wall ties, etc.);
- Performance testing of the concrete used (compression strength tests on samples taken after application).

Speed of Progress

Between 20 and 40 m² of wall treated per working day.

Handling singular points

Reserve openings for rainwater discharge.
Transitions with concrete riprap or other toe protection.

Cost Items

The cost of producing a dry sprayed shotcrete wall is between €500 and €1000 ex VAT per m^3 including all the suggestions made.

Variants on the technique or construction method

Variants in the framework of a contract may relate to the spraying method (dry or wet), the formulation of the concrete (addition of fibre or micro-silica) and any reinforcement and anchoring measures.

Shotcrete consolidation is a useful technique in many aspects (implementation, cost, ability to address a wide range of situations) but certain constraints (in particular aesthetic and heritage related) can lead to the use of other techniques.

In this case existing defective facing components are replaced (see sheet T5.8 for example - replacement of rubble).

It is also possible to install rigid structures (masonry, precast reinforced concrete, etc.) on the surface of existing facing (see case sheet 5.13)

Performance

Due to its good imperviousness and high frost resistance, dry-sprayed concrete generally has higher strength than ordinary concrete. Its also has a higher density than conventional concrete and therefore better durability.

References

NF EN 1990: Eurocode 0 - Basis for calculation of structures;
NF EN 1991: Eurocode 1 - Actions on Structures;
NF EN 1992: Eurocode 2 - Calculation of reinforced concrete structures;
NF EN 14487-1 2006 Sprayed Concrete: definitions, specifications and conformity;
NF EN 14487-2 2006 Sprayed Concrete: Execution;
ASQUAPRO (2010-2013). Technical guide “Sprayed Concrete”, 8 leaflets;
(1991) Clouterre Recommendations, Pont et Chaussées

3.2 8 Technique factsheet: Control matting made from Swiss wood wool (external erosion)

Authors: Lindner Suisse, Switzerland (holzwolle@lindner.ch)

Failure mechanism(s) and/or Basic function

External erosion on the water side.

Description technique

Slope erosion control matting reduces erosion by raindrops, surface erosion and the formation of rill erosion. The matting can be made from different materials, for example from biodegradable and natural fibrous materials, such as wood wool. According to the Swiss wood wool standard, wood wool consists of wood fibres that are 0.1–0.25 mm in thickness and 1.3–8 mm in width. The wood wool threads are up to 500 mm long and are felt-quilted together with a netting made of natural fibres.

Various types of Howolis wood wool erosion control matting are available, with the individual mix of fibres from different tree species playing an important role for the wood wool's durability, strength and stability (e.g. beech is less durable than fir or spruce). Timber species such as robinia, chestnut and larch have also already been tested and used.

The mats must function perfectly until the vegetation can assume their function (approximately 6–24 months, 2–3 vegetation periods depending on the location). Due to the fabric's excellent water retention capacity, good surface drainage and the niches between the fibres, wood wool erosion control mats improve the microclimate (moisture, temperature) for rapid vegetation establishment and reduce the risk of undercutting.

Applicability

The technique is applicable for: river levees, mountain stream levees, and regional levees.

Design

Design criteria

There are no design criteria for the installation of erosion control fabrics; rather the manufacturer's installation instructions are to be followed. It is important that the fabric is laid overlapping and free of tension. The tension generated by its own weight between the fixing points must not be higher than the fabric's tensile strength.

Which factors affect the dimensioning?

Criteria that affect the choice of type of matting (in total 7) are:

- Location: in the water, by the water, in the forest
- Steepness
- Ground conditions.

Figure 3.174 Detailed picture of wood wool erosion control matting

Figure 3.175 The low weight makes the non-woven erosion control easy to lay.
(Installation of an erosion control matting on a construction site.)

Other design considerations

Other considerations include mainly environmental considerations. The general advantages of the Howolis slope erosion control matting are the immediately effective protection of the soil surface, easy handling and the fact that the mats are completely biodegradable. Moreover, Swiss wood wool erosion control mats are made from local wood – certified with the 'Swiss Wood' label. They are a sustainable alternative to imported natural fibre variants such as coconut and jute and prevent introductions of unwanted exotic organisms. As a result, the product offers strong life cycle benefits.

Construction

During installation, it is important to ensure that no voids are created between the fabric and the ground. To this end, stakes can be used to affix the fabric to the slope (cuttings of willow species are best suited), 3–5 cm in diameter and 30–50 cm in length. Depending on the situation, appropriate seed is used before or after fabric installation to establish a vegetation cover. Pictures of the wood wool erosion control matting being applied are shown in Figure 3.174 and Figure 3.175.

3.2 9 Technique factsheet: Concreted riprap on the land side of levee (external erosion)

Authors: Thibaut MALLET, SYMADREM, Arles, France

For the corresponding case study, please refer to Section 3.1.10.

Failure mechanism(s) and/or Basic function

The technique is used against external erosion due to overflowing or overtopping.

Description technique

The landward side is reinforced with concreted 200 to 400 kg riprap so as to resist to high flow velocities, in case of overflow (Figure 3.177). The surface of the riprap is deliberately left rough to allow better dissipation of energy during overflow. Upstream and downstream of the spillways, the levees are set 50 cm above the millennial flood level to avoid a risk of circumvention in case of overflowing. A concrete beam, to precisely fix the overflow level and to avoid infiltration into concreted riprap, is realized on the levee crest (Figure 3.178). The concrete riprap is covered with topsoil to allow the structure to blend into the landscape (Figure 3.179).

A filter geotextile and a gravel layer are placed between concreted riprap and embankment (Figure 3.176). There functions are to ensure the water flow without internal erosion in case of watertightness failure within the embankment; to drain seepage up to the drainage outlet and to dissipate under pressures below the concreted rip rap layer.

The successive stages are illustrated below :

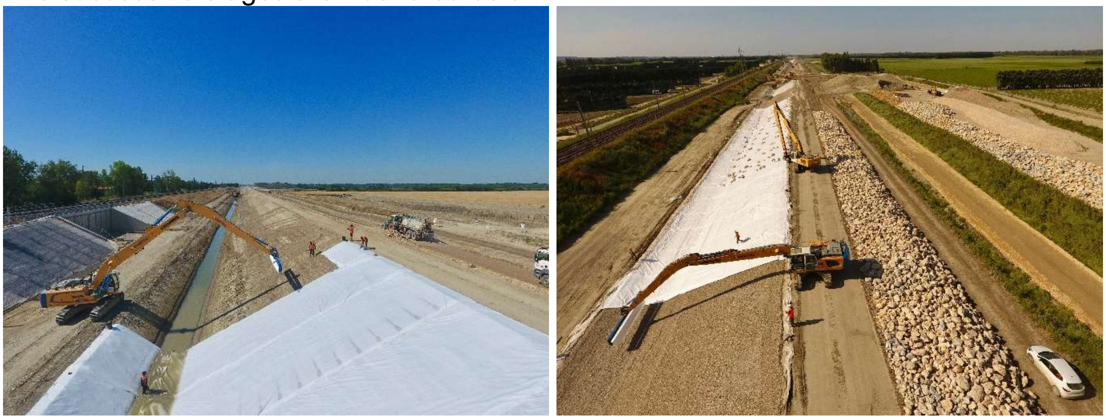

Figure 3.176 Stage 1 : Geotextile filter on embankment (left) – Stage 2 : gravel layer & second geotextile on gravel (right)

Figure 3.177 Stage 3 : rip rap on second geotextile (left) – Stage 4 concreting of rip rap (right)

Figure 3.178 Stage 5 : Concrete beam to determine the overflow level and to avoid seepages within concreted riprap

Figure 3.179 Stage 6 Landscape integration of concreted riprap (left) - Levee resistant to overflow in final stage (right)

The typical cross section of a levee resistant to overflow is below (Figure 3.180). It is followed by a picture illustrating the different components (Figure 3.181).

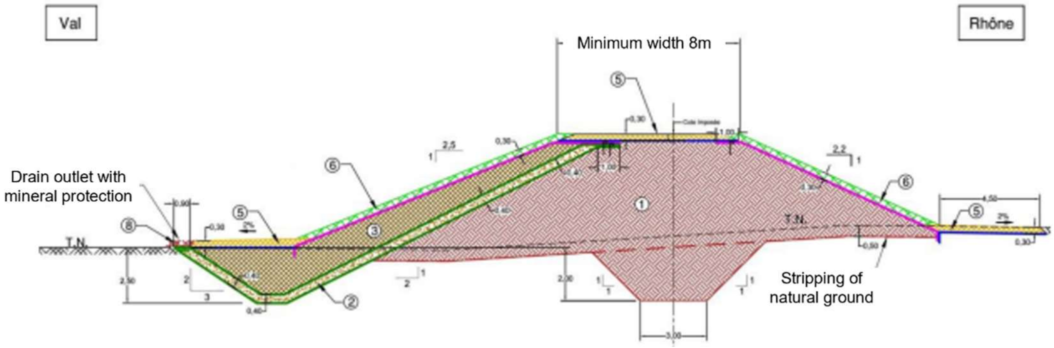

Figure 3.180 Typical cross section of a levee resistant to overflow in the Rhone delta

Figure 3.181 Building a levee resistant to overflow

Applicability

Concreted rip rap is applicable on river levees; coastal levees and estuarine levees. It must be carried out on a safe embankment designed to resist to internal erosion before overtopping. Concreted riprap on uncertain embankment, exposed to internal erosion, must be prohibited to avoid the safety feeling in order to avoid any false sense of security in the event of flooding.
Crossing pipes can be included in the structure with an appropriate treatment of the transition.
Implementation of levees resistant to overflow is usually done in rural areas to organize efficiently evacuation of population

Design

Design criteria

Two types of design must be considered: design of the hydraulic system and structure design. The design of levee systems can be done in a similar way to dams by choosing a crest level for the spillway corresponding to the protection level and by setting the crest level of sections not resistant to overflow at the level of exceptional floods (variable depending on the protected assets) plus a freeboard. The length of overflow-resistant sections is often conditioned by the impact on the water levels upstream, downstream or on the opposite bank of the river. The non-aggravation of water levels, which is one of the founding principles of French regulations on water structures, often requires the creation of long overtopping segments (5 km on each side in case of Rhone delta). For small protected areas, so as not to set the crest level of levees that do not resist overflow to exceptional occurrences of floods, some damage on these sections can be accepted, provided that the protected area has been pre-filled before overflow, so a sufficient mattress of water at the foot of the structure at the time of the spill on the unreinforced sections can prevent their breaching.

The design of structures can been carried out with formula of Isbach ; Knauss or Pinto (Rock Manual). Usually, design is carried out for riprap without concrete; concrete giving an additional safety. In case of Rhone delta, for levees with a 5 m height and an overflowing level of 30 cm for the safety level, 200/400 kg riprap appears an optimal blocometry to avoid costly structure and to facilitate concreting of rip rap.
Transition between levees resistant to overflow and not to resistant overflow must be treated by continuing the protection against overflowing for few metres on the levees not resistant to overflow

Which factors affect the dimensioning?

The non-aggravation of water levels, which is one of the founding principles of French regulations on water structures, often requires the creation of long overtopping segments (5 km on each side in case of Rhone delta).

How are key parameters for this technique Measured or estimated?

Controls of blocometry and concrete respectively are carried out according to conventional controls. For an efficient concreting of rip rap, a concrete sufficiently liquid is needed to favorize the migration of concrete in riprap.

Other design considerations

Table 3.13 design considerations

Uncertainty	***Influence on application of technique***
Increasing flood frequency, intensity or duration due to climate change.	All materials are durable. In case of increasing of flood frequency, overflowing could occur less than 100 years
Dryer periods possibility of cracking in embankments	Filter geotextile and gravel layer between embankment and concreted layer is designed to prevent from internal erosion Top soil reduces possibility of cracking
Land subsidence	Less than 1 mm/year Altimetry is controlled each 10 years
Requirements to make embankments warn before failure or to fail slowly	Filter for internal erosion
Material prices	Considered when evaluating different solutions Other components as treated soils with quicklime can be replaced the concreted rip rap for small levees (H < 2,5 m)
Environmental considerations	Top soil on concreted rip rap to integrate the structures in the landscape

Construction

Which methods of construction are available?

The methods of construction are classical methods used for major earthworks

Variants on the technique or construction method

Other techniques are possible (concrete, gabion mattresses, grid, treated soils with quicklime). Material availability, durability and robustness of design and construction must be the criteria taken into account for protection against external erosion throughout the lifetime of the structure.

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Degoutte G. & Tourment R., 2021 – Spillways on river levees – édition Quae https://www.quae.com/extract/3425

Mallet et al., 2013, Sécurisation des digues du Rhône entre Beaucaire et Arles : conception hydraulique du système et des digues résistantes à la surverse - Digues maritimes et fluviales de protection contre les submersions, 2ème Colloque national, Aix-en-Provence, 12-14 juin 2013 https://hal.science/hal-01544813

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Design manuals/standards/codes of practice

Rock Manual
https://kennisbank-waterbouw.nl/DesignCodes/rockmanual/

3.2.10 *Technique factsheet: External protection of embankment by weeding technique (external erosion)*

This contribution is based on: *FICHE TECHNIQUE Protection externe du talus par technique d'enherbement. External protection of embankment by weeding technique. From Recueil de méthodes et de techniques de confortement et réparation des digues de protection en remblai of the Comité Français des Barrages et Reservoirs (CFBR).*

Autors: W. BRASIER (CNR) ; B. CHALUS (CNR)
Contributors: C. MOIROUD ; M. SUTTER (INRAE) ; D. POULAIN (INRAE) ; P. LEDOUX
Reviewers: F. PEGUIN (EGIS) ; M. SUTTER (INRAE)
Final approval: A. RULLIERE (INRAE)

Failure mechanism(s) and/or Basic function

Primary functions:
Protection against external erosion caused by rainfall runoff (on both slopes) or by water flow along the water-side of the structure.

Secondary functions:
This surface protection layer can also provide other functions due to its design:
• Landscape integration
• Ecological diversification and enhancement (particularly for insect fauna)

Description technique

Traditionally, dikes have an external protection layer designed to resist stresses, mainly hydraulic, and generally constructed using civil engineering techniques (rock armoring, concrete, bitumen, etc.). Depending on the level of stress and the situation, it is possible to replace or complement this type of protection with ecological engineering techniques, such as grass cover.

This sheet focuses on the techniques implemented to achieve a grass cover adapted for the external protection of the slopes of a hydraulic structure.

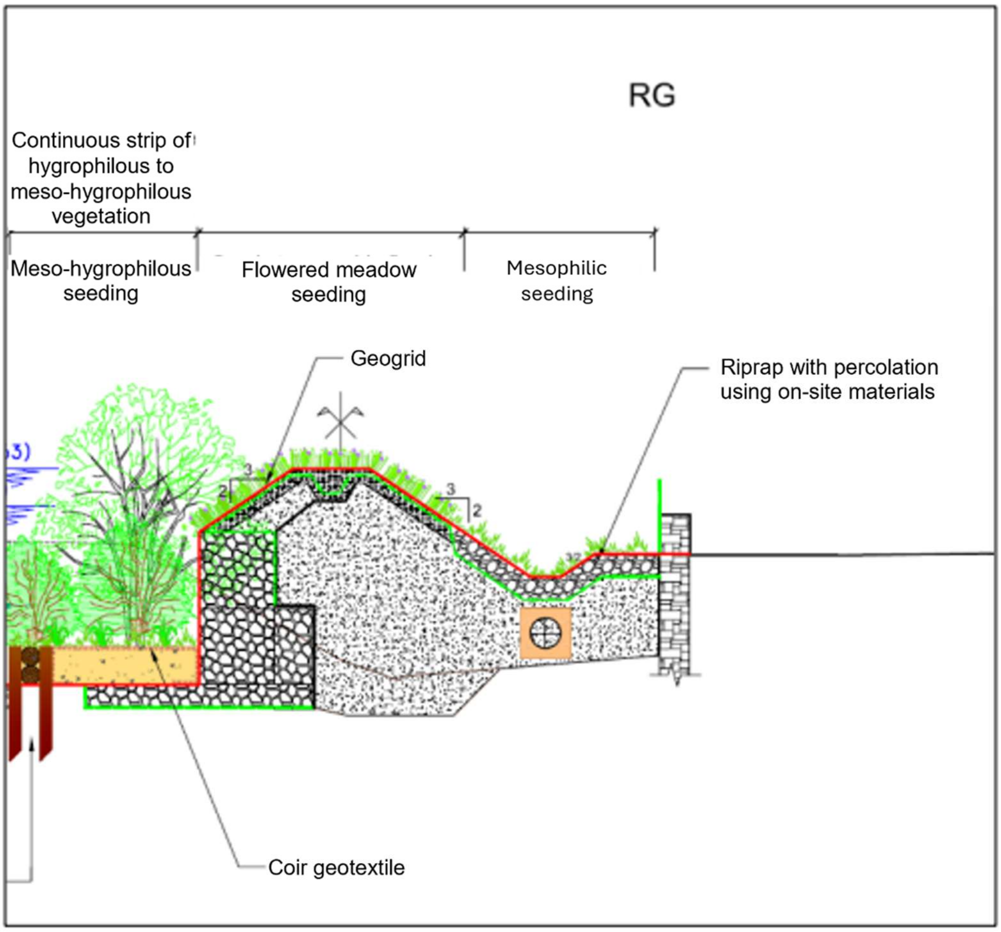

Figure 3.182: Example of slope protection (Source: CNR - Charbonnières)

Design

Design criteria

External erosion is caused by the following hydraulic stresses:
• Flow velocities during flood events
• Wave action during flooding
• Runoff erosion during heavy rainfall

The areas most affected by erosion are primarily the water-side slopes, which are progressively degraded due to soil detachment, particularly as flow velocity increases.

The external protection of the slopes must therefore form a system capable of resisting shear forces, through an appropriate selection of plant species.

In general, this type of protection also requires performance in terms of retaining fine materials (topsoil), which is made possible in the short term through the use of biodegradable geotextiles.

The dimensioning of grass-covered slopes is based on two main considerations:

- the choice of plant species (which will not be discussed in detail in this sheet, as it varies greatly depending on stresses, climate, etc.); case-by-case studies are necessary;
- the stability of the protection system, particularly before vegetation has grown.

The choice of plant species is based on criteria such as:

- Their biomechanical characteristics: they must ensure good root development and have a certain tolerance to maintenance operations (mowing, etc.).
- The origin of the plant material: it is necessary to use only vegetation produced or harvested near the work site. Species from other countries must be avoided, or at least examined carefully on a case-by-case basis.
- To obtain the most diverse mixture possible, seeds may be collected directly from the natural environment following the protocol of labels such as “Native Vegetation” or obtained from nurseries that produce seeds from local genetic stock.

Figure 3.183 lope protection with 732 g/m² jute mesh fabric – immediately after installation (Photo Credit: CNR – Charbonnières)

Figure 3.184 Slope protection with 732 g/m² jute mesh fabric – in spring after seed germination has started (Photo Credit: CNR – Charbonnières)

Figure 3.185 Protection of a small embankment slope using 900 g/m² coconut geotextile – immediately after installation (Photo Credit: CNR – Charbonnières)

Figure 3.186 900 g/m² coconut geotextile on a section of dike exposed to the watercourse – flood debris still visible (Photo Credit: CNR – Charbonnières)

Figure 3.187 Gabion–earth dike at Charbonnières – geogrid + 732 g/m² jute geotextile + seeding + climbing plants – side view (Photo Credit: CNR – Charbonnières)

Figure 3.188 Gabion–earth dike at Charbonnières – geogrid + 732 g/m² jute geotextile + seeding – top view (Photo Credit: CNR – Charbonnières)

Which factors affect the dimensioning?

Before using grass-cover techniques on protection structures, the context, the stakes, and the various hydraulic constraints must be thoroughly analyzed.
This requires a detailed understanding of the following information:

- Topographic characteristics (slopes, existing berms, etc.)
- Nature of the materials making up the bank
- Observable degradations, to be correlated with the "natural" condition of the dike
- Hydraulic conditions (flows, water levels, tidal variations, hydrology, wave action, etc.)
- Condition, type, and density of vegetation on the bank (from toe to crest)
- Uses and general context of the site
- Maintenance methods and associated costs for the surfaces

Other design considerations

- Timing of the intervention: this must be aligned with the optimal growth period of the selected vegetation species, in order to keep the slope "bare" for as little time as possible.
- Availability of local vegetation resources (availability, quality, distance).
- Accessibility of the site during implementation.
- Conditions for maintenance and/or upkeep.

Application

For new structures, structural reinforcements, or dikes whose slopes are bare, the aim is sometimes to establish vegetation capable of protecting the slope: grass cover with carefully selected species.

This vegetation is not suitable for all situations, as strong currents and impacts can degrade it very quickly. For this reason, these techniques are mainly used in river environments in "calm" zones subject to gradual rises and falls of floodwaters.

Depending on the constraints, different techniques can be applied as shown in Figure 3.189.

Technique	CONSTRAINTS: Property width <5 m	5 m>	Substrate: Boulder	Gravel Sand	Clay Silt	Bank slope: Gentle 10/1 à 3/1	Steep 2.5/1 à 1,7/1	Battering	Maximum speed: < 0.5m/s	0.5<v< 2m/s	2<v<3m/s	Blockage	Tidal range: None	<1m	> 1 m
Vegetation technique															
Helophytic seeding	O	O	N	O	O	O	O	O	O	N	N	N	O	O	O
Terrestrial seeding	O	O	O	O	O	O	O	O	O	O	O	N	O	O	O

O ou N — Construction feasibility / Yes (O) – No (N)

O — Construction possible only with protection

Figure 3.189 The physical criteria related to each technique (source: CNR)

Variants on the technique or construction method

There are many plant species capable of providing this type of protection; it is essential to select them carefully in order to obtain durable protection in all seasons. A case-by-case study must be carried out depending on local conditions (sun exposure, soil type, etc.).
Furthermore, it should be noted that the design of this surface protection depends on its position on the structure. As a general rule, the following principles can be applied:

- Protection against erosion on the upper part of the dike slope or on the slopes on the protected side, when these areas are rarely submerged, and in the short term (the time needed for vegetation to develop), is achieved using jute-type and/or biodegradable geotextiles, suitable for more terrestrial environments.
- The middle part of the slope, which is more exposed to surface erosion and to erosion caused by the river discharge, requires more robust surface protection techniques. The use of biodegradable coconut-fiber geotextile is recommended. It is sometimes made even more robust through reinforced fastening using interwoven metal wires, which hold the geotextile tightly against the ground using wooden stakes, especially in concave areas of the longitudinal profile of the dike (intrados).
- The lower part of the slope is the most exposed to various river-related stresses: water level fluctuations, wave action, and flow velocities. It is preferably treated using mixed techniques that combine mineral protection and bioengineering solutions.

There is no technique as "lightweight" as a grass-covered slope; however, any heavier external protection technique can replace it effectively when there are no specific biodiversity concerns or cost constraints.

Construction

The typical sequence of work on a site is as follows:

- re-profiling of the embankment body (if needed);
- installation of a soil-anchoring geotextile (optional depending on local hydraulic conditions and slope gradients);
- spreading of at least 20 cm of topsoil if not already present;
- installation of biodegradable geotextile (jute, coconut fibre, etc.) if short-term reinforcement is needed before grass establishment;
- implementation of herbaceous seeding by hydroseeding on large surfaces, supplemented by manual seeding in less accessible areas. A careful soil preparation is required (fine, shallow raking carried out manually and/or mechanically to prevent seeds from sliding down to the lower part of the protection structure, removal of waste, roots, stones, etc.) before carrying out the seeding operations.
- Maintenance and monitoring to be planned: mowing to densify the sowing, watering during heatwaves if necessary.

Which methods of construction are available? What materials are required?

- Seed mixture containing species with strong root systems: especially fescue species due to their high resistance to pull-out forces.
  Species must be chosen according to the site's biogeoclimatic conditions. Experience shows that this mixture should have a fairly high specific diversity (10–15 species). Additionally, for biodiversity purposes, it should include approximately:
  - 85% grasses
  - 5–10% legumes
  - 5% other plants ("flowering species"): nectar-producing species, etc.
- To support vegetation, topsoil or silty alluvium should be used over 20 cm, with the following (ideal) composition, to be adapted to local conditions:
  - Coarse elements: gravel (2 mm to 2 cm): less than 30% dry weight;
  - Fine elements:
    - coarse sands (0.2 to 2 mm): 30–35%;
    - fine sands (0.02 to 0.2 mm): 10–15%;
    - silts (0.002 to 0.02 mm): 30–35%;
    - clay (< 0.002 mm): 4–6%;
  - Organic matter: 2–3% (silty river alluvium) or 3–15% (topsoil);
- If medium- or long-term stabilization is required, the following products may also be used:
  - jute geotextile fabric (typically 732 g/m²) on terrestrial-type slopes (upper slopes or slopes outside the flow);
  - coconut fibre geotextile fabric (typically 740 g/m² or 900 g/m²) on watercourse banks;
  - three-dimensional geogrid.

Other construction considerations?

Environmental impact

Environmental points requiring attention include: noise, dust related to site traffic, accidental pollution, waste management, and fauna/flora protection. Furthermore, intervention periods are generally restricted to take into account wildlife breeding seasons and optimal plant growth periods. For herbaceous species, for example, suitable time windows generally range from September/October to March/April.

Execution controls

It is necessary to ensure the conformity of the plant material. The following elements must be verified:

- the quality of geotextiles to be used, if necessary;
- the origin of seeds. It is preferable to accept only plant material produced or harvested near the site (local strains);
- compliance with standards related to seeding.

Final inspections

- validation through test plots (mixtures tested in a nursery);
- supply chain control.

In addition to standard execution control, such protection requires regular monitoring and maintenance to remain effective.

Singular points in the construction

Near cables, structures, etc., manual implementation is preferred. Moreover, the proximity of these specific points may affect the behaviour of the planted species and will require special attention: growth inhibited or encouraged due to shading, water accumulation, e

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3.2.11 Technique factsheet: Repair of embedded stone pitching (external erosion)

This contribution is based on: *FICHE TECHNIQUE 7.5 Réparation de perré maçonné. From Recueil de méthodes et de techniques de confortement et réparation des digues de protection en remblai of the Comité Français des Barrages et Reservoirs (CFBR).*

Main authors: E. VUILLERMET (BRLi)
Contributors: M. SUTTER (INRAE) ; D. POULAIN (INRAE) ; R. TOURMENT (INRAE)
Reviewers: JF-FREZET (Egis); S. PATOUILLARD (DREAL)
Final Approval: JF-FREZET (Egis) ; S. PATOUILLARD (DREAL)

Failure mechanism(s) and/or Basic function

Primary Functions
The main functions of embedded stone pitching, depending on their environment, are the following (these functions are not systematically all sought):

- Resistance to external erosion on the water side (maritime environment and river environment);
- Retaining wall on the water side (maritime environment and river environment);
- Retaining wall on the land side (maritime environment and river environment);
- Resistance to external erosion on the land side or overflow (rather in a maritime environment, river to a lesser extent);

Secondary Functions
The embedded stone pitching provides a upstream sealing function (although imperfect in the long term) which must be associated with a drainage system, particularly on the Protected Area side.

It can also provide a function of skin stability, or even of superficial protection of a thin main component providing sealing function.
On the valley side, we can add a function akin to the stabilization of stiffened slopes.

It should be noted that there is also "dry stone" masonry, i.e. without hoarding. However, this type of masonry systematically requires an association with other techniques to satisfy the desired functions (for example a geotextile filter, a drainage system, etc.).

Description Technique

The term "embedded stone pitching" as used in this sheet, refers to structures made of a composite material comprising large elements (masonry stones, rough rubble or bricks), joined by a mortar (the joint and the hoarding). They form one of the components of a structure and can be of very varied design: single wall, double wall, facing etc. (Figure 3.190).

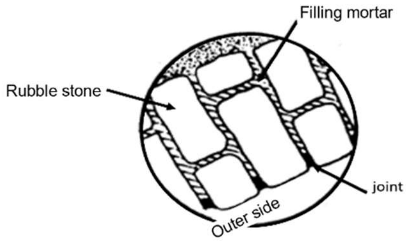

Figure 3.190 Embedded stone pitching - river side slope (Photo credit: BRLi) and cross-section of the various components (Source: BRLi)

These structures are found in both the river and maritime sectors and can present a wide variety of disorders/pathologies:

- Pathologies of the constituent materials:
  - Rubble: erosion, abrasion, dissolution etc.
  - Mortar: dissolution, scaling, spalling etc..
- Disorders of the masonry structure: general stability, differential settlements, localized collapses, displacements, scour at the toe, rubble heaving, cavities, disorders at the base layers etc.

This sheet focuses on the repair of such structures, which often have to be integrated into the landscape/architecture in a particular way because they are old (before the end of the 19th century).

Performance to be Achieved

The performance to be achieved depends on the functions provided by the embedded stone pitching.

- For external erosion, the pitching and its components must withstand various possible forces (tensile forces, abrasion, etc.), resulting from the action of waves, fast flowing overflow or floating objects.
- For the mechanical strength of the slope, resistance to soil and water pressure is required. A certain height may also be needed to prevent overflow from a defined project flood.
- For impermeability, the facing must limit the seepage of water through the dike but, unlike dams, without requiring perfect impermeability since the hydraulic loading is not permanent.

These different performance requirements are achieved by:

- The overall design of the pitching.
- The size and quality of rubble stone.
- The quality of the laying.
- The quality and durability of masonry binding mortar and pointing mortar.
- Addition of extra components in addition to the pitching (drains, filters, reinforcing geotextiles).

Use

Embedded stone pitching is a technique rarely used for building a dike, but repairing such structures on old dikes is a frequent operation. Such repair usually involves replacement of rubble, re-pointing of joints or complete replacement of embedded stone pitching on an existing structure.

The “large components” form the bulk of the mass of the embedded stone pitching. Sometimes due to their state of degradation, it is necessary to correct their shape or replace them. They are generally made of resistant materials and arranged to minimise voids (filled in with mortar): rubble, cut stones, bricks and so forth.

There are two types of mortar used on embedded stone pitching:

- mortar to bind the rough masonry (or structural joints), used in repair when it is necessary to reinforce the structural functions of the structure (make it monolithic, mechanical strength of pitching);
- joint mortar on the facing (between 3 and 5 cm deep) used to seal and protect the masonry mortar below.

Variants

In terms of the materials used, there are different variants:

- Injection of epoxy resin to fill in cracks (especially in rubble stone);
- Polyurethane foams to improve imperviousness;
- Water repellent coatings to improve sealing or protect the facing.

Design

Principles

The design of embedded stone pitching itself is hardly practised nowadays and the approach is to restore it to how it was originally while retaining or stiffening the stone-mortar structure.

The approach used and composition of the mortar will differ depending on the function of the joint:

- Structural masonry joints, which are internal joints, used to give the masonry a certain monolithic structure. The mortar therefore requires significant compression strength and shear strength.
- Since the facing joints essentially act as protection against external attacks and ensure the continuity of the facing, they must have tensile strength, shear strength and generally a high degree of imperviousness.

The rubble, masonry stones or bricks are chosen for their mechanical characteristics and their aesthetic appearance. Given that these works were generally constructed in the past, it is common for the original quarry (located near the site) to be unknown or worked out. To maintain an equivalent aesthetic appearance, the rubble in place is generally re-used and the repair is based on use and modification of the mortar.

In addition to these approaches, it is generally necessary to perform calculations on the overall stability of the pitching. These calculations are similar to those performed for masonry retaining walls. They will refer to:

- The guides prepared by STRESS (NATIONAL UNION OF CONTRACTORS SPECIALISING IN STRUCTURAL REPAIR AND REINFORCEMENT) in the series “Concrete and Masonry Structures”;
- Research by:
  - Benjamin Terrade “Structural evaluation of masonry retaining walls”, 2017 - University Paris-East - Ifsttar.
  - Marie Bisoffi-Sauve “Study of masonry stone structures using the discrete element method: characterisation and modelling of the cohesive behaviour of joints”, 2016 - University of Bordeaux.

Required Parameters

The data needed to design repairs is data from a survey on:

- The condition of the rubble: degradation of blocks by alteration or the freezing-thawing cycle;
- The condition of the joints depending on their nature (lime, lime cement mortar, grout, etc.) and their function (structural or surface pointing);
- Status of the underlying support: presence of underlying cavities due to undermining, presence of wall ties etc.
- Information on the reasons for the damages: ageing of joints, disintegration of rubble, subsidence due to loss of underlying materials, deformation under action of plant growth, circulation of water due to internal erosion;
- The presence of components associated with the existing pitching (weep holes, inspection openings, penetrations).

This rather qualitative data is generally supplemented by quantitative data, particularly where specific functions are involved: permeability if drainage if required, mechanical parameters (friction angle...) if a stability calculation is being performed and so forth.

Design Elements

- Local stone resources (availability, quality, distance from quarry) if identical reconstruction is desired and stones are missing. If there is no local resource, it is common practice to extract the rubble and replace it as well as possible while compensating for irregularities by injecting mortar; or it is re-used after being carefully extracted.
- Preparatory joint-cleaning work: different equipment is used depending on the joint, from air chisels to high-pressure hoses. The aim is to remove degraded and non-adherent materials, to provide a minimum thickness for the new pointing cement, to have clean substrates to guarantee good adhesion etc. This phase requires “washing” the substrate.
- Preparatory work on substrates: reconstitution of the underlying layer, which may for example be a sand-gravel cement (no draining character but limits settlement), a sand-gravel material, or “as dug” gravel 0-20.
- Access to the site during work and construction time constraints (especially in a maritime environment due to swell and tides).
- Environmental conditions: throughout the period of work and more particularly during the preliminary cleaning work.
- Architectural integration in relation to the constraints on sites classified as a Historical Monument.

Construction

Which methods of construction are available?

Different construction methods can be used (Figure 3.191 shows elements for choosing the method):

- Caulking, which consists of manually packing joints to a significant depth and which must be followed by traditional grouting;
- Manual traditional grouting;
- Spraying methods: mortar conveyed by pipe is sprayed with compressed air and sprayed into the joints.
- Grouting methods.

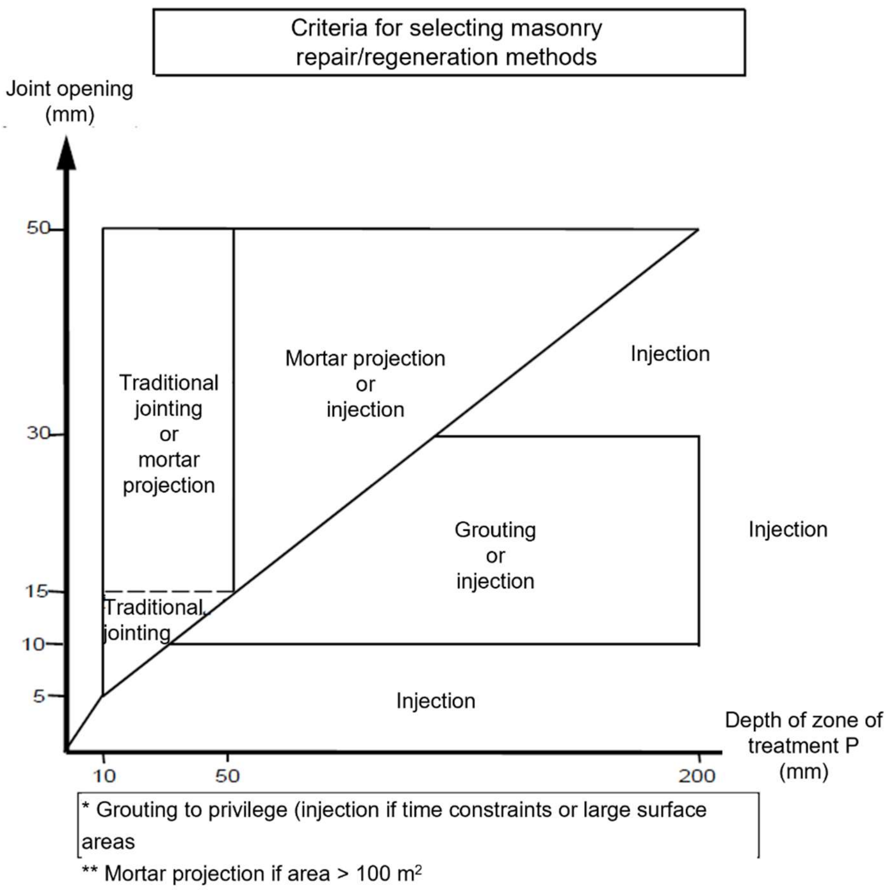

Figure 3.191 Choice of methods for repairing masonry joints (source: CETMEF)

Besides this, different choices of mortar are available:
The mix used for re-pointing mortar is essentially a function of the depth that has been lost, the opening of the joints, the type of large components, working constraints and are related to the function of the joint.

For example, the following mix proportions may be used:

- Proportions for lime cement re-pointing mortar using the traditional method (closing a joint to a shallow depth): 400 kg to 500 kg of cementitious material per m3 of sand with 1/3 cement and 2/3 hydraulic lime;
- Proportions for lime cement caulking mortar: up to 600 kg cementitious material per m3 of sand with 1/3 cement and 2/3 hydraulic lime;
- Proportions for mortar installed dry or wet (filling of joints to a significant depth): the proportion of cementitious material for the mortar once it has been inserted must be of the order of 400 to 500 kg/m3 of sand.

Also by way of example, the mix proportions in Table 1 were used for restoring partially buried pitching with re-pointing using a coating machine equipped with a gun.

Table 3.14 example of cement mixes for embedded stone pitching.

PARTS OF THE WORK	Class of concrete or mortar	Consistency of fresh concrete or mortar	Characteristic strength FC 28	Minimum amount of cement	Maximum aggregate size
Re-pointing of masonry out of water = visible surfaces	Lime cement mortar	S2 –S3	20 MPa	200 kg/m³ cement 42.5 + 300 kg of hydraulic lime XHN or XHA	0 / 3 or 0 /6
Re-pointing of buried masonry (fluctuating ground water level)	Cement mortar	S2 – S3	20 MPa	500 kg/m³ cement 42.5	0/3 or 0/6

For the specifics of mortar components and for specific mixes, the reader is invited to refer to the standards and recommendations listed at the end of the sheet. It is also possible to use ready-to-use mortar which provides better control of uniformity of the products used, provided that the proportion of water stipulated by the manufacturer is followed.

NB: Depending on the mix, installation techniques (manual, semi-manual or mechanical) are matched to the situation. Therefore these two aspects need to be considered in parallel when selecting a mix. To validate the choice, a test area is always carried out. In particular, the compatibility between the rough masonry structural mortar and the re-pointing mortar must be checked.

For stability calculations, limit equilibrium or finite element methods and models will be used in accordance with the geometry of the structure to be analysed. It is common practice not to study the stability when repairing embedded stone pitching. The only justification sometimes expected is that of the number of weep holes to be put in place to drain the new pitching effectively.

Other construction considerations?

Specifications of the material

The supply is specified in the Specific Technical Clauses listing the expected characteristics for:

- Rubble/stone/brick;
- Type of joint;
- Joint mortar: lime cement mortar / cement mortar depending on the location of joints (see table below);
- The nature of the cementitious material also depends on the aggressiveness of the water and what the structure is exposed to (e.g.: marine environments require the use of suitable cementitious material to limit sea water attack on cement (attack of lime, attack of C3A component in cements).

It also defines the principles for removing stones to replace them.

The particular features of installing joints are defined: not during freezing temperatures or during very hot weather or drought and ideally between 5 and 30°C. Hydraulic conditions (water level fluctuations, flooding, etc.) should also be taken into account.

Worksite Execution

Several steps are involved in the work to repair embedded stone pitching:

- Partial or total unpacking of old joints and mortars (Figure 3.192):
    - Joints are removed manually or mechanically. The aim is to remove as many joints as possible while maintaining the structure of the masonry. Depending on parameters such as cohesion or adhesion of the joint, unpacking will be carried out more or less at depth, while avoid damaging stones, bricks or rubble. In principle, existing joints are unpacked to a minimum depth of approximately 2 to 2.5 times the thickness of the joint.
- If necessary, a structural mortar will be installed.
- Preparation of joint substrates: before the joint is put back it is necessary to clean the contact surfaces of the joint with the other components of the embedded stone pitching. This cleaning consists of removing dust and brushing the interfaces to remove any elements which may interfere with adhesion of the joint.
- Re-pointing: to ensure good adhesion of the new seal, the substrate must have sufficient moisture for the mortar to take properly. It may be necessary to moisten (by spraying) these substrates or, on the contrary, to limit runoff water on the surface of the pitching (Figure 3.193).
- Setting of the facing grout.

Notes:

- During construction work, the different steps of unpacking and re-pointing must be carried out progressively and without destabilising the structure (with shoring up if necessary). To ensure this small surface areas are renovated at a time.
- If there is a fluctuation in water level, it is essential to take changes in water level (internal and external) into account during the concreting stages, so that it can set when the pitching is immersed.
- In a maritime environment, lime-cement mixes should not be used because of the influence of seawater on these materials.

Figure 3.192 Cleaning the facing and cleaning out joints (photo credit: BRL(i)

Figure 3.193 Site installing new joints - Injection pump and air gun (Photo credit: BRLi)

Figure 3.194 Photograph of a renovated facing (Photo credit: BRLi)

Equipment

Cleaning of surfaces is necessary to remove traces of moss, calcite, dirt, or vegetation to perform the work on a clean substrate. It can be carried out using different equipment and methods (Table 3.15) requiring skilled labour so as not to damage existing materials.

Table 3.15 Methods and equipment for cleaning surfaces for work on embedded stone pitching

Dry with light sand blasting and brushing	High pressure variable sanding booth (pneumatic sandblaster) - very fine or enriched sand - Personnel protection
Sandblasting with water and brushing	High pressure cleaner with hydro sandblasting kit - brush on HP cleaner
Hydropneumatic sanding by pressurising a mixture of sand and water - (Abrasion cleaning)	High pressure pump and hose

Removing existing mortar requires different equipment depending on the consistency and depth to which the mortar is to be removed. In general the following equipment is used:

- Hammer and chisel.
- An air chisel.
- An abrasive disk.
- Pressurised water (about 10 to 20 bar).

There are also different methods for re-pointing, presented in

Table *3.16*.

Table 3.16 methods and equipment required for re-pointing during work on embedded stone pitching

Method	Equipment required
Traditional manual method and caulking.	Manual jet and filling with cat's tongue trowel. Finishing if necessary with sponge and brush.
By spraying or injection (dry or wet).	Screw pump or pressure tank. Air gun.

Note: For this type of work where labour costs are much higher than the cost of supplies, ready-made High Performance jointing mortars are preferred over mortar mixed on site. They guarantee better uniformity of the product and ease of use.

Disturbance / Environment

- Management of wash water can be done by installing a geotextile at the toe of the structure for filtration before it is discharged, a gutter to channel the water to a holding area and leave it to settle or filter it afterwards.
- Control of dust by incorporating water: wet spraying, blowing with air/water mixture, washing with water rather than air.

Inspection of the Work

- Product approval on the basis of a product data sheet.
- Validation of Quality Assurance Procedures and construction work procedures.
- Validation on a test area: methodology and colour.
- Monitoring of work execution: filling of joints.
- Inspection of construction of the reconstituted underlying layer (if relevant).
- Finishing work: brushing of projections.
- During windy weather or under strong sunlight: wetting of joints or curing compound.

For more details see "Re-pointing maritime masonry structures, CETMEF", for which we present Table 4, Summary of Inspections.

Speed of Progress

Very variable depending on the state of degradation of the pitching and the resources that can be used. Some manual methods or means are stipulated by the architect or required by the limited space available.

Handling of singular points

Since pitching consists of rubble or stones of similar but not identical geometry, a large number of singular points have to be considered. The method of consolidating or repairing pitching therefore considers these singular points, generally compensated for by adding mortar. The mortar is of course designed to adhere properly to the various interfaces (masonry, penetrations, soil etc.).

Reinstatement of the Site

Re-pointing makes stones adjacent to the joint dirty. It is therefore necessary to clean them when the new joint is finished.

Table 3.17 From the Quick Reference Card: Re-pointing of maritime masonry structures - CEREMA West- April 2018.

Phasing	Points to be checked	Control actions
Suitability test	Cleaning – repointing – finishing – curing – cleaning on test panel; verification of conformity to the Quality Assurance Plan in terms of results and resources (effectiveness, no alteration of stones, depth of raking, condition of joint backing, joint colour)	Visual and documentary checks (material reception
Joint raking	Depth of raking – complete removal of deteriorated areas	Visual
	Stability of the structure during works	Visual inspection, instrumentation
Preparation	Cleanliness and moistening of the joint backing	Visual (moist but without runoff)
Repointing	Composition and characteristics of the mortar	Consistency and compression tests on specimens
		Sample collection for records
	Homogeneous filling of the joint, shape of the joint	Visual
Finishing	Exterior appearance, colour	Visual
General cleaning	Cleanliness of the masonry facings	Visual

Source : STRRES et Cerema

References

Rejointoiement des ouvrages maritimes en maçonnerie, (Re-pointing of maritime masonry structures) CEREMA - 2017.

Rejointoiement des ouvrages en site maritime » ("Re-pointing of maritime structures"), Nicolas Bourneton, Jacques Billon, Michel Menguy, Benoît Thauvin, Laurent Riou, XIIèmes Journées Nationales Génie Côtier – Génie Civil - Cherbourg, 12-14 juin 2012 (XII National Coastal Engineering Days - Civil Engineering - Cherbourg, 12-14 June 2012).

Ouvrage de maçonnerie, juin 2006, Sous-direction des Monuments Historiques et des Espaces Protégés (Masonry Structures, June 2006, Historical Monuments and Protected Areas Branch)

Guides Cerema/CETMEF dans le domaine des aménagements fluviaux (Cerema/CETMEF guides in the area of river hydraulic works).

Guides Cerema/CETMEF en technique portuaire (Cerema/CETMEF guides on port technology).

Ouvrages d'art (ex. fascicules CCTG pour les murs et ponts en maçonnerie et guides IQOA) (Infrastructure (e.g. General Technical Clauses leaflets for masonry walls and bridges and IQOA guides)).

Professionnels (artisans bâtisseurs en pierres sèches) (Professionals (craftsman dry stone builders)) http://www.pierreseche.fr/lire/index.php?rubid=14

Eurocodes 6

As well as the documents from which these guides were written:

- Standards and Recommendations for Constituents of mortar:
- Cement:
- Aggregate:
- Mortar: NF EN 998-2: definition and specifications of masonry mortars
- Mixing water:
- Ready to use products
- Leaflet 64, General Technical Clauses
- NF P95-102: Repair and reinforcement of concrete and masonry structures - sprayed concrete.
- NF P95-107: Repair and reinforcement of masonry.

3.2.12 Technique factsheet: Revetment and berm using granular material (slope stability)

Authors: Edina Koch, Richard Ray (Széchenyi István University), Zsombor Illés, General Directorate of Water Management, Hungary

For the corresponding case study, please refer to Section 3.1.11.

Failure mechanism(s) and/or Basic function

The technique improves the sliding stability of the landside slope.

Description technique

A stabilizing revetment that acts as a drainage filter provides the simplest solution. A berm or flattened slope further supports the existing levees on the landside (this is how modern embankments have "evolved"). Engineers prefer free-draining granular soil for repairs so the embankment remains sufficiently stable during persistent flooding. A significant advantage is that the revetment construction does not require the demolition of the slope; it only requires the preparation of the existing levee for connection. It requires a large volume of high-quality material, which may not be available. This solution may serve as the final step or provide additional support to the temporary slope stability berm used during flood protection (Figure 3.195).

The free-draining revetment provides stability through the **confinement** of the **blanket layer,** while reducing the risk of hydraulic failure due to uplift or internal erosion and piping.
Adding additional coarse-grained material increases the stability of the levee. When the revetment is wide enough, it provides additional slope stability due to its surcharge weight and negative driving moment

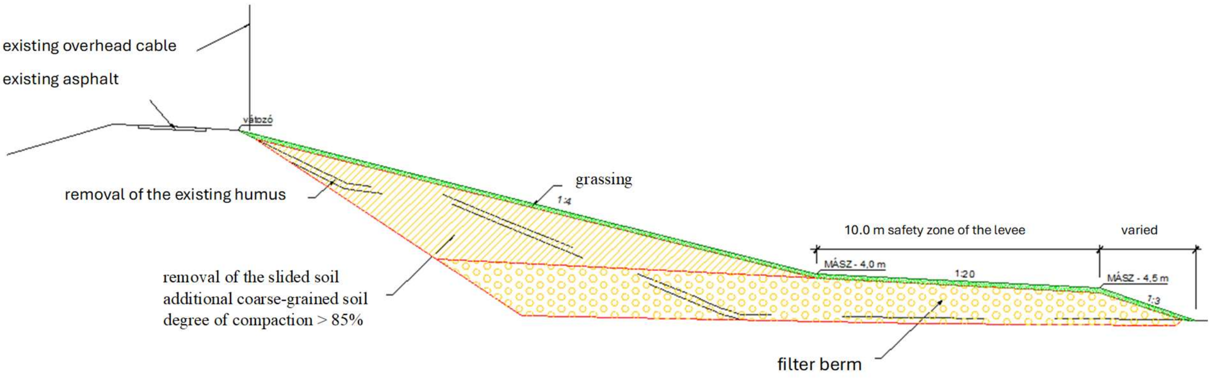

Figure 3.195 Cross section of the reinforcement containing stabilizing granular soil.

Applicability

Construction of a revetment using granular material reinforces the levee when there is enough space and the slope of the existing levee is steeper than 1:3. The technique is suitable in rural areas.

The subsoil may consist of fine-grained blanket materials (clay or silt), and even uplifting of the blanket on the landside due to high pore water pressure may occur.

Design

Design criteria

The design follows primarily an ultimate limit state approach. The Hungarian national annex to Eurocode 7 specifies design approach 3 for overall stability problems. This is particularly reasonable for levees, because partial factors are typically applied to soil strength parameters rather than actions. The partial factor used for shear strength parameters is 1.35. For characteristic water levels on the waterside, the standard flood level (MÁSZ) given in Hungarian governmental regulations serves as an estimated maximum stage following EC7-1 Section 2.4.5.3.(1)P. The level used in the design is typically MÁSZ+1.0m. Analyses have shown that this level corresponds to a return period of about 1000 years.

Which factors affect the dimensioning?

The main design parameters for overall slope stability are the internal friction angle and cohesion. The hydraulic conductivity of the levee and the subsoil also affects the seepage and pore pressure conditions, which impact the effective soil strength.
Proper design requires numerical modeling to assess the interaction between soil strength, seepage forces, and effective confinement conditions in the levee. Such modeling requires expert knowledge to generate reliable results.

How are key parameters for this technique measured or estimated?

Conventional methods, e.g. boreholes, CPTu soundings, geophysical measurements, laboratory tests can be used to determine the input parameters for both analytical and numerical models.

Other design considerations

considerations related to:

- increasing flood frequency, intensity, or duration due to climate change;
- land subsidence;
- environmental considerations;

Construction

Which methods of construction are available?

Conventional earthwork technologies can be applied.

Other construction considerations?

The available site dimensions may limit the feasibility of increasing the levee width.

Variants on the technique or construction method

Fine-grained material may be used for the revetment as an alternative to coarse-grained material. However, this may result in reduced shear strength and lower hydraulic conductivity.

References

Design manuals/standards/codes of practice

EN 1997-1 (2004). Eurocode 7: Geotechnical design - Part 1: General rules [Authority: The European Uni-on Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

Galli L.: Az árvízvédelem földműveinek állékonysági vizsgálata. Országos Vízügyi Hivatal. Budapest, 1976. (in Hungarian)

MSZ-15292:1999 - Árvízvédelmi gátak biztonsága. Magyar Szabványügyi Testület, Budapest, 1999. (in Hungarian)

Publications

Koch, E., and Ray, R.P (2024). Effect of flood duration on levee stability. *Proc. XVIII ECSMGE-2024, Portugal*. DOI 10.1201/9781003431749-314

3.2.13 Technique factsheet: Increase of subsoil strength with vacuum consolidation (slope stability)

Authors: Kaj Althuis MSc, Cofra B.V. and Jeroen Dijkstra MSc, Cofra B.V., the Netherlands

For the corresponding case study, please refer to Section 3.1.12.

Basic function

For over 70 years, vacuum consolidation has been a successful technique in global projects. Vacuum consolidation distinguishes itself by applying an additional load to the subsoil through the reduction of atmospheric pressure in the vertical drain, rather than applying a traditional surcharge. The vacuum pressure serves as an equivalent to surcharge loading during the consolidation of the subsoil. Consequently, the combined effect of a surcharge and a vacuum pressure significantly accelerates soil consolidation and effectively reduces residual settlement and horizontal deformation. Vacuum consolidation can also be used to create strength in the compressible layers without applying a surcharge load on top of the surface.

Description of technique

Vacuum consolidation systems vary, but all share the requirement of a closed system where the vacuum pressure is intended to target the cohesive soil layers exclusively. This is achieved through one of three methods (further explanation below Figure 3.196).

- Encasing the area with a liner to create a sealed environment. (see left in Figure 3.196).
- Installing horizontal drains, connected to vertical drains, below the groundwater level inside the compressible cohesive soil layers, and connected directly to a pump. (see center in Figure 3.196)
- Individually connecting each prefabricated BeauDrain-S drain (section of drain connected with a vertical tube), eliminating the need for an underground horizontal drain and crossing any non-cohesive layer on top of the material to be treated. This method accommodates variations in the thickness of the top layer and is effective even with thicker top sand layers. (see right in Figure 3.196)

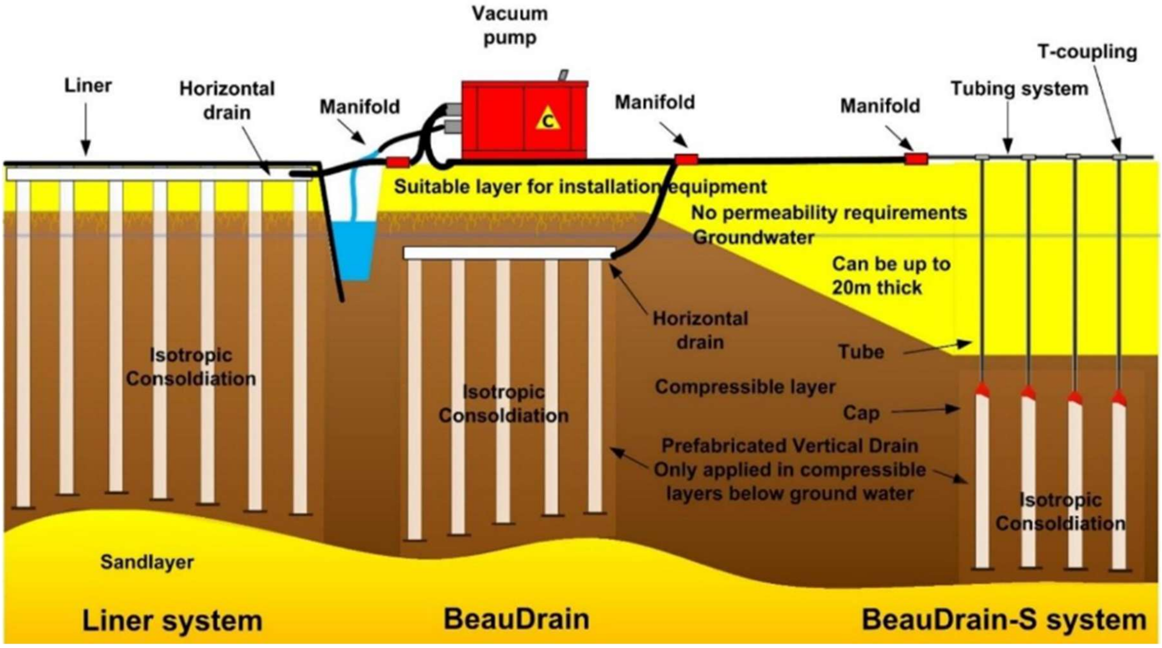

Figure 3.196 Cross section of vacuum consolidation techniques

Liner vacuum system

In this approach, horizontal drains are installed at ground level (after the installation of traditional vertical drains). The entire area is then covered with liner. The liner is placed below the groundwater level and into the sealing layers on all sides, this way a sealed system is created in which vacuum can be applied under the liner.

This method's advantages include reduced leak risks and the elimination of drain and tube prefabrication, proving particularly effective in high groundwater conditions and larger project areas.

BeauDrain

The standard BeauDrain system entails a system in which screens of vertical drains connected to a horizontal drain, installed below the groundwater inside the compressible layer, are connected to vacuum pumps. The system is installed using a specially designed plough, pulled through the soil by a base excavator.

The system can be applied on loose platforms with sand thicknesses up to 1.5m and ground water down to 2m below the platform.

BeauDrain-S

An evolution of the standard BeauDrain, the BeauDrain-S can be installed using conventional PVD equipment. It comprises a vertical drain attached to a predetermined length of polyethylene tube, which is designed to run across the thickness of the sandy working platform and, depending on groundwater levels, extended into the sealing layer to ensure the vertical drain is sealed, only applied in the compressible layers and below te ground water.

Following installation, the polyethylene tubes, trimmed at platform level, connect via couplings to form strings. These strings then link to a vacuum pump at the field's edge through a collector tube. The system can be installed with any sand layer thickness and ground water level (max 6m below the surface to have an effect).

Advantages of the technique

Vacuum consolidation offers three primary benefits over traditional methods:

- Less Surcharge Required: Generating up to 70kPa of effective pressure lowering, the vacuum system can substitute for up to 3.5m of sand surcharge. (Note that this is depending on method, ground water table and soil conditions. Some projects have only 30kPa of effective pressure due to the local boundary conditions)
- Shorter Consolidation Time: The additional load from the vacuum accelerates soil consolidation, expediting project completion, when also surcharge is used.
- Stability of Surroundings: Traditional surcharge methods can induce lateral pressure and potential slope stability. Vacuum consolidation, requiring less sand, reduces this deformation risk and promotes more compact soil through direct isotropic pressure lowering. This is pictured in Figure 3.197.

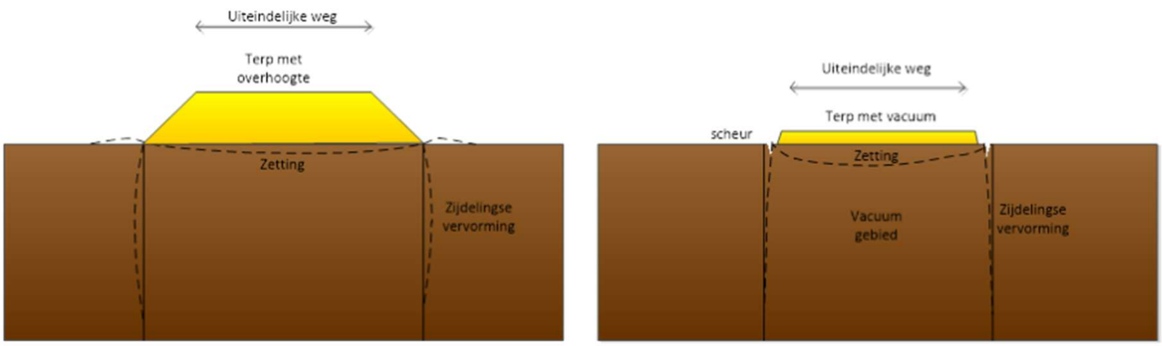

Figure 3.197 Lateral deformation, left: traditional consolidation with surcharge, right: vacuum consolidation.

Design

The design of a vacuum system can be performed in the same manner as a traditional vertical drain design. The effective vacuum inside the soil can be modelled as an equivalent surcharge. The effective vacuum in the soil is a function of pump pressure, minus the difference between the water-vacuum connection inside the pump and the ground water level, and the flow losses in the system.

Strength gain can be determined using SHANSEP theory, taking into account the effective vacuum inside the ground and degree of consolidation.

Design criteria

In reality the subsoil is never completely homogeneous. A vacuum consolidation design must therefore also be adjusted based on the local soil profile. This is essential in order to create a good working system. There are several scenarios that greatly increase the likelihood of achieving a good vacuum pressure in the system, see the table below for these scenarios along with an explanation and how to mitigate the risks.

When the scenarios as given in Table 3.18 are taken into account during the design phase, it is possible to eliminate the potential problems before construction and create an optimal vacuum pressure in the soil.

Table 3.18 Considerations during design phase

Ideal scenario	Explanation	Possible mitigating measurements
Little variation in the soil profile	If there is little variation in the soil, there is less chance of short-cutting the vacuum with the air or have sever inflow of water into the system. A lot of variation increases the chance that a thicker top sand layer is missed in the design.	Additional soil investigation should be carried out to better map the subsoil. Or an extensive slurry wall should be installed around the vacuum system
No historical activities at the location	Historical foundation piles, old dike breaches, or old excavations increase the chance of losing the vacuum due to heterogeneous soil conditions.	Old elements should be removed as much as possible, or they should be mapped in detail. A liner vacuum could in this case be more optimal
The area should be as compact (square) as possible	Each pump has a maximum distance at which it can create enough vacuum pressure when there is a combined air-water transport. The most efficient scenario is when the pump is in the middle and can work in all directions and water air transport is limited as much as possible.	If the project area is elongated, more pumps per square meter are needed than in a square area. A detailed pump plan must be made.
No stiff clay layer at the surface	When stiff layers are present at the top, the drain installation holes remain open or get filled by the material of the working platform. This increases the risk for air inflow	A liner vacuum system should be chosen in this scenario
High groundwater table	The vacuum pump has to pump the water from the groundwater table upwards; if this height distance is small, there is minimal pressure loss. Each meter of height difference lowers the effective vacuum by 10kPa	The pump should be placed as low as possible; it can even be buried to minimize the distance to the groundwater table.

No sandy intermediate layers	Intermediate sand layers can cause the vacuum to connect to the air or large water inflow which affects the sealed system. This will cause pressure loss.	Additional soil investigation should be carried out to better map the subsoil.

Conclusion

Vacuum consolidation has several major advantages over traditional consolidation with vertical drains:

- Less surcharge required
- Less consolidation time needed
- Less influence on the stability of the surroundings

However, to take advantage of these benefits, the system must be installed correctly. If there is a connection between the drains and the air or somewhere within the system, the vacuum pressure will be lowered and the system will work less effective. In the previous chapter, several scenarios were mentioned in which vacuum consolidation is very suitable. If one of the scenarios is not applicable, measures should be taken during the design phase to reduce the chance of a system being installed incorrectly. A well-executed design will subsequently result in a good working vacuum system. It is advised to contact a specialist to review the design made.

3.2.14 *Technique factsheet: Bamboo-mattress embankment reinforcement (slope stability)*

Authors: Hendra Jitno, National Institute of Technology (Itenas), Bandung, Indonesia

For the corresponding case study, please refer to Section 3.1.13.

Failure mechanism(s)

When embankments are constructed over soft clay, the weight of the fill increases the stress within the soil and generates pore water pressures that continue to accumulate over time. Under these undrained conditions, the soil behaves similarly to a viscous fluid, potentially leading to bearing capacity failure if the embankment is constructed too fast.

As excess pore pressure dissipates, the soil volume decreases, leading to consolidation settlement. This process involves a gradual transfer of stress, which can take a significant amount of time and plays a key role in influencing potential failure mechanisms, such as progressive failure. During this stage, additional strain may develop even after the soil has reached its plastic state.

Using the bamboo-mattress technology, some of the vertical loads are taken by the structural strength of the bamboo-mattress, while the foundation layer is gaining strength as the pore pressures slowly dissipate. During this process, the embankment loads will be slowly transferred to the foundation layer. This technique assumes that the bamboo mattress will deteriorate over time, leaving the soil to withstand the embankment loads alone.

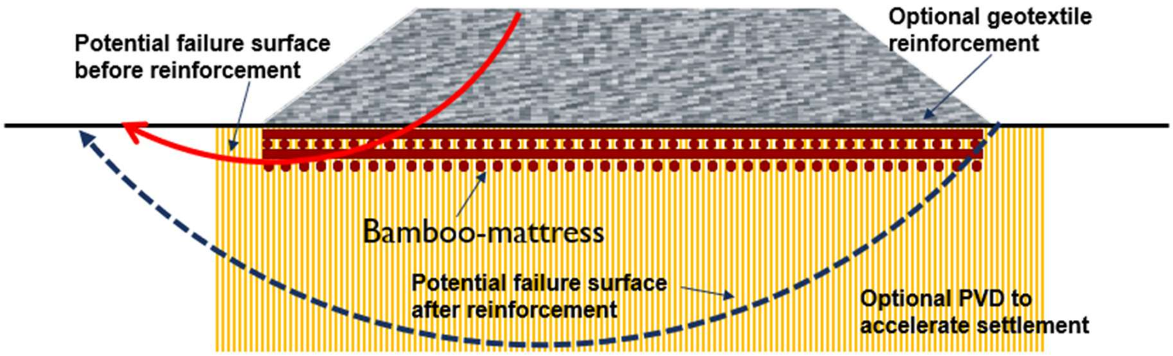

Figure 3.198. Failure mechanism of the embankment improved by bamboo-mattress reinforcement.

Figure 3.199. Construction of embankment improved by bamboo-mattress reinforcement.

The benefits of the bamboo-mattress ground improvement include:

- It enforces the failure surface not to pass through the middle of the embankment
- Distribute the embankment load more uniformly
- Reduce differential settlement due to its high stiffness
- Provides buoyancy
- Cost-effective, and the bamboo is available locally.
- Environmentally friendly and sustainable.

Design challenges:

- How to prevent bearing capacity failure during construction?
- How to minimise the long-term settlement, which includes both primary and secondary consolidation.

Description of the technique

This factsheet describes the bamboo-mattress ground improvement. Below is step by step construction of this technique:

1. Site Preparation

- Clearing and Grubbing: Remove vegetation, topsoil, organic material, and debris from the embankment footprint.
- Excavation (if needed): In cases where weak surface soil needs to be replaced or lowered for levelling, shallow excavation is performed.
- Levelling: The subgrade is roughly levelled to prepare a flat base for placing the bamboo mattress.
- Platform (optional): In cases where the embankment is constructed on swampy areas, a layer of platform is needed to work on.

2. Bamboo Mattress Fabrication

- Selection of Bamboo: Mature, seasoned bamboo poles (typically 6–10 cm in diameter) are chosen for strength and durability.
- Mattress Size and Spacing:
    - Poles are laid longitudinally and transversely, spaced typically at 20–30 cm intervals.
    - The arrangement resembles a woven or grid mesh, tied securely with rattan, wire, or plastic straps.
- Mattress Thickness: Varies with design; can be single or double layers depending on load and soil conditions.
- Prefabrication or On-Site Assembly: The mattress may be assembled on-site or nearby and transported to the embankment base.

3. Placement of the Bamboo Mattress

- Positioning: The assembled mattress is carefully laid flat on the prepared subgrade. In soft soils, bamboo poles may also be driven slightly into the surface to secure the mattress.
- Anchoring: Edge anchoring or staking may be done to hold the mattress in place and avoid shifting during fill placement.
- Overlap (if multiple panels are used): A minimum overlap of ~30 cm is provided between adjacent mattress sections to maintain continuity.

4. Placement of Separation/Filter Layer (Optional but Recommended)

- Geotextile Layer: A non-woven geotextile is placed over the bamboo mattress to act as a filter and separator, preventing the migration of fine subgrade particles into the fill material.
- Sand Blanket (optional): A thin sand layer (10–20 cm) may be used above the geotextile to protect it during fill placement.

5. Placement of Embankment Fill

- Initial Lightweight Fill: The first fill layer (up to 30–50 cm) is carefully placed to avoid overstressing the subgrade. This may consist of sand, light granular soil, or lightweight material.
- Stage-wise Filling:
    - Fill is placed in thin layers (~30 cm thick) and compacted properly.
    - As the fill height increases, careful monitoring of settlement and lateral displacement is conducted.
- Vertical Drains (if required): Prefabricated vertical drains (PVDs) may be installed through or beside the mattress to accelerate consolidation.

6. Monitoring and Surcharge Loading (if applicable)

- Instrumentation: Settlement plates, inclinometers, and piezometers may be installed to monitor performance.
- Surcharge Loading: Temporary additional loading may be applied at the crest to accelerate consolidation if required by design.

7. Final Embankment Construction

- After the foundation has stabilized (with or without surcharge), the embankment is raised to its final height.
- Proper compaction and quality control are maintained throughout to ensure uniform strength and minimal settlement.

8. Finishing Works

- Slope Protection: The embankment slopes are stabilized using:
    - Turfing (grass cover)
    - Riprap
    - Gabions
    - Bioengineering methods
- Drainage Installation: Surface drains, toe drains, and side ditches are constructed to channel surface and seepage water safely.

9. Maintenance and Inspection

- Regular inspections are conducted to check for signs of:
    - Surface cracking
    - Excessive settlement
    - Erosion or slope failure
- Maintenance involves repairing damaged bamboo edges, reinforcing weak sections, and improving surface drainage.

Applicability

The bamboo-mattress reinforced embankment technique is especially suitable for areas with soft, weak, and compressible subsoils, where conventional embankment construction would pose stability, settlement, or bearing capacity problems. The most appropriate applications of this technique include:

1. Road and Railway Embankments on Soft Ground
    - Ideal for rural or peri-urban areas where transportation routes cross swampy, silty, or clayey soils.
    - Provides a stable base and minimizes differential settlement, ensuring long-term serviceability.
2. Flood Control Levees and River Dykes
    - Useful for constructing or reinforcing levees in flood-prone regions with weak alluvial soils.
    - Enhances stability against seepage and lateral sliding during high water levels.
3. Coastal and Tidal Area Embankments
    - Well-suited for low-lying coastal zones, deltas, and tidal flats with soft marine clays.
    - Bamboo resists biological degradation better than many organic materials under wet conditions (though treatment may be needed for long-term durability).
4. Agricultural Infrastructure
    - Used for farm road embankments, irrigation canal bunds, or raised platforms in waterlogged paddy fields.
    - Cost-effective for rural development projects due to use of local materials.
5. Temporary or Low-Volume Access Roads

- Supports temporary roads or construction access tracks in soft terrains where rapid, low-cost construction is required.
- Suitable for logging roads, oil palm plantation access, or temporary construction camps.

6. Foundation Support for Rural Structures
   - Can be used under lightweight rural buildings or platforms where soft ground conditions exist and shallow foundation support is needed.
7. Environmentally Sensitive Areas
   - Bamboo, being a natural and renewable material, makes this technique suitable for use in ecologically sensitive zones where minimal environmental disruption is required.

Design

Design criteria

The following design criteria are generally adopted for the design:

1. Settlement and Consolidation Control Considerations.
   - This is generally applicable to highway projects, where settlement is crucial to ensure the safety of vehicles. Long-term settlements must be within serviceability limits.
   - The rate of consolidation is also prescribed in the design criteria, which necessitates the use of PVD to accelerate pore pressure dissipation.
2. Stability (Global and Local)
   - Stability criteria are applicable for short-term and long-term conditions. The target FOS for long-term conditions is 1.50, and for short-term conditions is 1.30.
   - Progressive Failure Check is also recommended for embankments constructed on sensitive clays where gradual strain accumulation may trigger failure.
3. Hydraulic and Drainage Considerations
   - Seepage and Uplift: Ensure that water pressures under the bamboo mattress do not significantly reduce the effective stress.
   - Filter/Separation Layer: A non-woven geotextile is recommended above the mattress to prevent contamination of fill material and preserve drainage.
   - Surface Drainage: Crest and slope drains must be installed to prevent erosion and maintain embankment stability.
4. Construction Considerations
   - Staged Construction: Embankment fill must be placed in layers (≤ 30–50 cm) with monitoring of settlement and stability.
   - Instrumentation: Settlement plates, inclinometers, and piezometers should be used in critical projects.
5. Applicable Standards:
   - Local geotechnical design standards (e.g., ASTM, British Standards, or Indonesian SNI guidelines)

Which factors affect the dimensioning?

The dimensioning of a bamboo mattress-reinforced sea-dyke is influenced by a combination of geotechnical, hydraulic, structural, and environmental factors, as follows:

1. Geotechnical Conditions
    - Soil bearing capacity: Determines the allowable pressure from the dyke and mattress, which control maximum height of the embankment.
    - Subsoil stratigraphy: Soft soils may require thicker mattresses to distribute loads.
    - Pore water pressure and drainage: Influences stability and potential for slope failure.
    - Settlement and consolidation behavior: Affects long-term performance and mattress deformation.
2. Structural and Material Considerations
    - Bamboo strength and durability: Bamboo quality, species, and treatment against rot/pests influence design life and load capacity.
    - Mattress thickness and width: Must resist uplift, wave impact, and provide sufficient anchoring.
    - Tying or lashing methods: Affects integrity and resistance to disintegration under hydraulic loads.
    - Geotextile or filter layer usage: Prevents soil piping and increases stability.
3. Slope Geometry and Dyke Design
    - Height and slope angle of the dyke: Steeper slopes may require more robust reinforcement.
    - Crest width and side slopes: Influence load distribution and overtopping resistance.
    - Transition zones and toe protection: Require careful design to prevent undermining or erosion.
4. Environmental and Ecological Factors
    - Salinity and biodegradation: Bamboo durability is affected by exposure to saltwater and biological agents.
    - Vegetation interaction: Roots can help reinforce but also affect water flow paths.
    - Habitat considerations: The mattress should avoid disrupting local ecosystems.
5. Construction and Maintenance Constraints
    - Local availability of bamboo and labor: Determines practicality and cost.
    - Ease of installation and anchoring: Influences design modularity and size.
    - Maintenance strategy: Periodic checks may be needed to assess bamboo degradation or damage.

How are key parameters for this technique measured or estimated?

Similar to other geotechnical projects, the soil parameters are determined with conventional methods (borings, cone penetration tests and laboratory measurements). Analyses can be done using Finite Element or Finite Difference Method, which incorporates soil-structure interaction.

Other design considerations

Designing an embankment with bamboo-mattress reinforcement involves several uncertainties due to the variability in material properties, soil behavior, construction practices, and environmental conditions. These uncertainties can affect both the stability and long-term performance of the embankment. Below is a comprehensive list of uncertainties associated with this type of embankment design:

1. Soil-Related Uncertainties

- Spatial variability of subsoil strength (e.g., undrained shear strength of soft clay)
- Inaccurate or insufficient geotechnical investigation data
- Soil stratification and thickness variations
- Rate and magnitude of primary and secondary consolidation
- Pore pressure dissipation rates
- Sensitivity and strain-softening behavior of soft clays
- Soil permeability and drainage characteristics
- Anisotropy in shear strength or stiffness
- Uncertainty in modeling progressive failure in soft soils

2. Bamboo Material Uncertainties

- Natural variability in bamboo strength and stiffness
- Durability of bamboo over time, especially in wet or biological environments
- Effectiveness of bamboo treatment (e.g., boron, creosote) against decay and insect attack
- Uncertain lifespan of bamboo under actual field conditions
- Creep and degradation of bamboo under sustained loading
- Quality of workmanship in tying and assembling bamboo mats

3. Load-Related Uncertainties

- Magnitude and distribution of embankment fill loads
- Traffic or surcharge loads (if the embankment supports roads or facilities)
- Hydraulic loads from seepage, floodwater, or tidal influences
- Inaccurate estimation of construction equipment loading during fill placement
- Changes in loading conditions during service life

4. Construction-Related Uncertainties

- Quality and consistency of fill material placement and compaction
- Deviation from design during bamboo mattress installation
- Uneven settlement or improper leveling of subgrade
- Damage to bamboo mats during placement or filling
- Lack of skilled labor or traditional construction knowledge
- Unplanned delays leading to bamboo exposure and premature degradation

5. Hydraulic and Environmental Uncertainties

- Unanticipated rainfall intensity and stormwater runoff
- Rising groundwater levels or tidal backflow
- Variability in erosion or scour potential at toe or slope
- Salinity or chemical attack accelerating bamboo deterioration
- Impact of climate change (e.g., increased rainfall, flooding, or temperature extremes)

6. Structural and Stability Uncertainties

- Uncertain contribution of the bamboo mattress to load distribution

- Inadequate factor of safety due to underestimated pore pressure buildup
- Potential for localised failure (e.g., slip surface at mattress interface)
- Uncertainty in the actual boundary conditions used in slope stability modelling
- Difficulty in predicting long-term performance and deformation under creep

7. Monitoring and Performance Uncertainties

- Unavailability or inaccuracy of field instrumentation data (settlement plates, piezometers, etc.)
- Uncertainty in interpreting monitoring data to trigger staged loading or remedial measures
- Unexpected differential settlement or lateral spreading

8. Model and Analytical Assumptions

- Simplified assumptions in analytical models (e.g., linear-elastic behavior, homogeneous soils)
- Uncertainty in applying conventional slope stability methods to reinforced bamboo systems
- Limitations of bearing capacity models for bamboo-mattress foundations
- Potential mismatch between theoretical design and real behavior

9. Long-Term Maintenance and Degradation

- Uncertainty in the rate of bamboo decay or loss of structural contribution over time
- Maintenance challenges in detecting and repairing subsurface bamboo damage
- Future changes in land use or loading not accounted for in the original design

Construction

Which methods of construction are available?

For swampy or coastal areas, there are limited options available. The general construction method would be :

- Placement of the Bamboo Mattress
- Placement of Separation/Filter Layer, such as a geotextile layer and sand blanket
- Placement of Embankment Fill
- Monitoring and Surcharge Loading (if applicable)
- Final Embankment Construction

Variants on the technique or construction method

Generally, the construction method does not differ that much.

References

Design reports on Semarang-Demak Integrated giant sea dyke with toll road (unpublished)

3.2.15 Technique factsheet: Vertical drain system in embankment body (slope stability)

Authors: Edina Koch, Richard Ray (Széchenyi István University), Zsombor Illés, General Directorate of Water Management, Hungary.

For the corresponding case study, please refer to Section 3.1.14.

Failure mechanism(s) and/or Basic function

The technique of a drain system using granular material improves the sliding stability of the landside slope.

Description technique

Trencher machines provide a longitudinal drainage system filled with coarse gravel (Figure 3.200). Soil characteristics of the subgrade and the groundwater level determine the system's depth. The drains usually do not penetrate the permeable aquifer layer. The equipment dimensions and capacity dictate the trench geometry. Drainpipes placed at the bottom of the longitudinal trenches direct the collected water to controlled exit points. Usually, a smaller free-draining stabilizing berm provides additional landside toe support (Figure 3.201).

Figure 3.200 Trencher machine

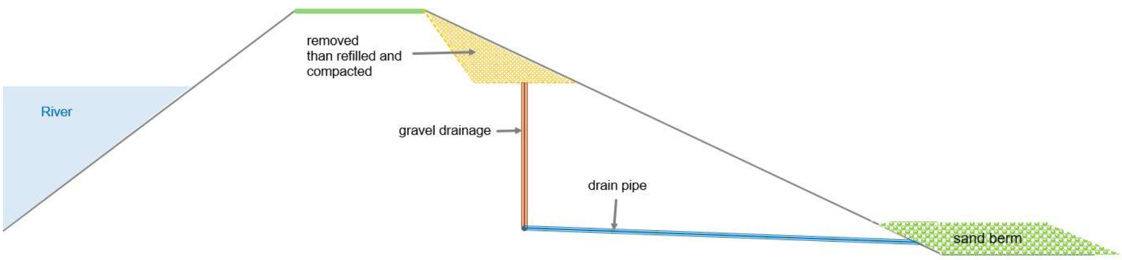

Figure 3.201 Schematic cross section including vertical gravel drainage and small landside sand berm

This technique will mitigate seepage through the levee and increase its landside slope stability. Using a coarse-grained drainage system might control the seepage in the levee body. This system reduces water pressures and lateral seepage forces in the levee (on the landside). The drains quickly drain water from the levee body and prevent the reduction of the levees' strength.

Applicability

Constructing a drainage system using granular material reduces seepage forces and increases effective confining stresses throughout the embankment. The system requires minimal additional space for exit drains and is suitable where embankment widening is impractical. The technique is ideal in rural areas.
The subsoil may consist of fine-grained blanket materials (clay or silt).

Design

Design criteria

The design is primarily an ultimate limit state approach. The Hungarian national annex to Eurocode 7 specifies design approach 3 for overall stability problems. This is particularly reasonable for levees because partial factors are typically applied to soil strength parameters rather than actions. The partial factor used for shear strength parameters is 1.35. For characteristic water levels on the waterside, the standard flood level (MÁSZ) given in Hungarian governmental regulations serves as an estimated maximum stage following EC7-1 Section 2.4.5.3.(1)P. The level used in the design is typically MÁSZ+1.0m. Analyses have shown that this level corresponds to a return period of about 1000 years.

Which factors affect the dimensioning?

The main design parameters for overall slope stability are the internal friction angle and cohesion. The hydraulic conductivity of the levee and the subsoil also affects the seepage and pore pressure conditions, which impact soil strength.

Proper design requires numerical modelling to assess the interaction between strength, seepage forces, and effective confinement conditions in the levee. Such modelling requires expert knowledge to generate reliable results.

How are key parameters for this technique measured or estimated?

Conventional methods, e.g., boreholes, CPTu soundings, geophysical measurements, and laboratory tests provide the input parameters for analytical and numerical models.

Other design considerations

considerations related to:

- increasing flood frequency, intensity, or duration due to climate change;
- land subsidence;
- environmental considerations;

Construction

Which methods of construction are available?

There are different solutions for constructing the drainage system (trench). The classical hydraulic excavator with a backhoe and the trencher machine are the most used equipment.

Other construction considerations?

The primary consideration of the technique is the embedded depth of the drain and the height of the collected water outlets. An important aspect of the method is the coarse gravel fill material wrapped in a geotextile filter.

Variants on the technique or construction method

Different depths and different types of material can be used.

References

Design manuals/standards/codes of practice

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3.2.16 Technique factsheet: Retaining by means of soil nailing (slope stability)

Authors: Jan Couck (Flemish Government, Geotechnics Division, Belgium), Robin Lievens (Flemish Government, The Flemish Waterways, Belgium)

For the corresponding case study, please refer to Section 3.1.15.

Failure mechanism(s) and/or Basic function

When nailing, both the internal and external stability of the slope must be calculated. The nails must be deep enough to prevent an external slip plane. The grout around the nails must provide sufficient friction to prevent pullout. The steel of the nails must be strong enough to avoid breakage due to tensile stress.

Description technique

During nailing, rods are drilled into the ground. In this case hollow, self-drilling bars were used. At the end is a drill bit. The bars are secured with grout:

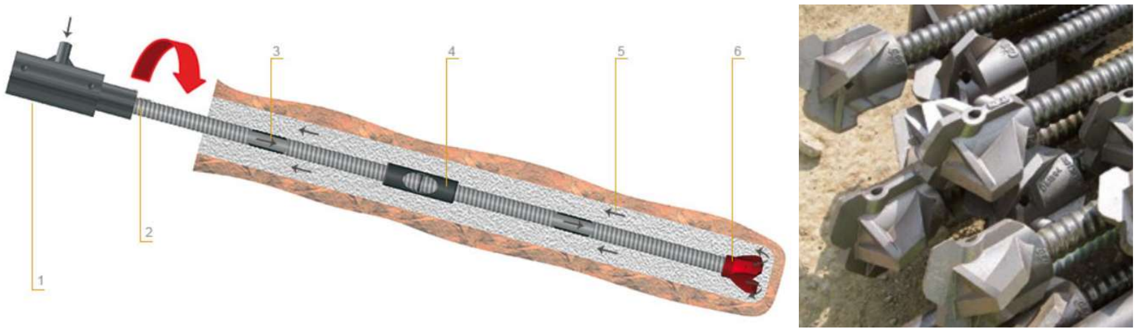

Figure 3.202 Details of the nails containing rods (source: website Dywidag)

Below are photos of the nail execution:

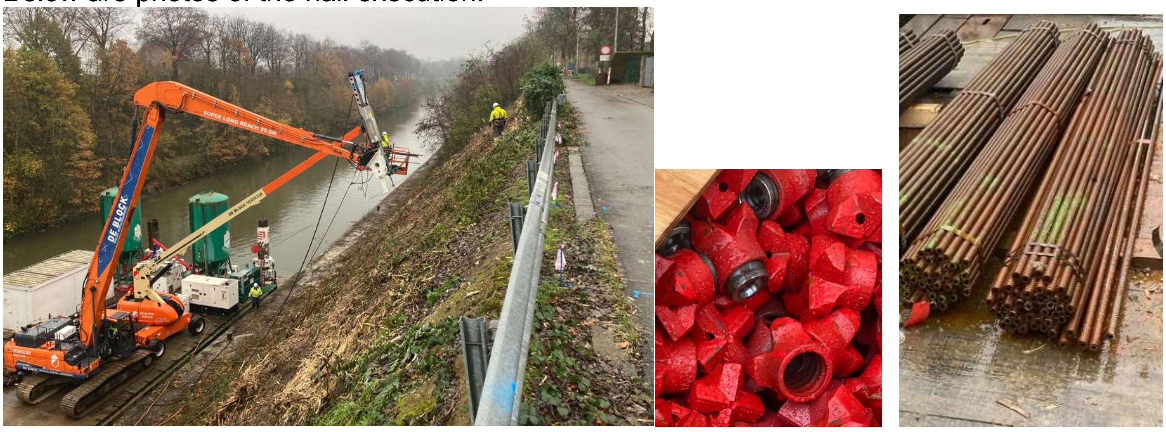

Figure 3.203 Photos of the execution of installing nails

The properties of the nailing are:

- Nails of types H0360-32 and H0210-32, with an anti-corrosion additional coating of 4 mm on the diameter.
- Drilling diameter of 100 mm.
- Four to five rows of nails with a vertical spacing of 2.5 m and a horizontal spacing of 2.5 m.
- The length of the upper rows of nails is 7 m; the length of the lower rows of nails is 10 m.
- Steel end plates with a nut at the end of the nails.
- A continuous steel wire mesh of 3 mm/80 mm on the front face of the slope, clamped by the end plate of the nails.

Gravel and stones in sand layer B can make drilling the ground nails difficult, requiring a special drilling head with hard bits.

It is necessary to perform preliminary tensile tests of the ground nails to test the geotechnical pull-out force. These tests are included in the specifications as a separate item.

Applicability

Nailing can be applied in various environments, as long as the nail head is above the groundwater. In marine environments, corrosion can occur more rapidly due to the influence of salts. Providing good corrosion protection for nails is challenging. In sandy soil, drilling nails is preferred. Drilling nails in very hard clay or petrified sand layers requires specialized drilling equipment and experience from the drill master.

Design

Design criteria

The design follows an ultimate limit state approach. The stability of the slope must be ensured throughout its entire lifespan.

Corrosion of the steel nails, couplings, nuts, and nail heads is a critical concern to guarantee the expected lifespan.

The contractor Herbosch-Kiere was responsible for conducting the detailed calculation of the nailing and also carried out the work.

Which factors affect the dimensioning?

For the design of nailing, the geometry of the slope, groundwater level, and soil properties are crucial.

The spacing between nails (horizontal and vertical), their length, steel quality, diameter, and bore diameter must be dimensioned, usually through iteration.

How are key parameters for this technique measured or estimated?

The soil parameters are determined with conventional methods (cone penetration tests). From these methods also load – deformation behavior of soils for Finite Element calculations can be derived. Behavior of the nails are derived from product specifications.

There are several methods (Clouterre, Bustamante, EC) for calculating the friction of grout.

Other design considerations

A work platform must be placed against the slope for the drilling equipment, taking up part of the canal.

Pay attention to underground utility lines when drilling.

Check whether the nails that reach private parcels are legally permitted.

Construction

Other construction considerations?

Nailing causes little vibration and noise.
Excess grout must be removed after drilling nails.

Variants on the technique or construction method

In this project, the nails were drilled and injected under low pressure. The grout injection can also be done under high pressure (200 bar). Steel wire mesh was used for the slope finishing. Other options include concrete slabs or geogrids. The nailed slope looks like this; the grass has not yet germinated:

Figure 3.204 Picture of the slope after reinforcement, including steel wire mesh

References

Design manuals/standards/codes of practice

Norm NBN EN 14490 'Execution of special geotechnical works - Soil nailing' from August 2010. French norm NF P94 270 from 2009 about soil nailing (in French).

Publications

POVM Anchoring Techniques is a publication by POV Macrostability from September 2019. This focuses on dike anchoring and is available at https://www.hwbp.nl/binaries/hoogwaterbeschermingsprogramma/documenten/rapporten/2020/12/17/publicatie-pov-vernagelingstechnieken/publicatie_POVM-Vernagelingstechnieken.pdf (in Dutch).

The Standard Specification SB260 of the Flemish government (in Dutch).

3.2.17 Technique factsheet: Anchored sheet pile wall (slope stability)

Authors: Theo Stoutjesdijk, Esther Rosenbrand and Meindert Van, Deltares, the Netherlands

For the corresponding case study, please refer to Section 3.1.17.

Failure mechanism(s)

The technique is used against slope stability of the inner slope.

Description technique

This factsheet describes the anchored sheet pile wall. The anchored sheet pile wall is installed in the crest of the dike as a measure against slope stability of the inner slope of the embankment. The sheet pile wall in combination with the anchoring and the soil of the dike provide the necessary safety. The strength is derived from the embedment of the sheet pile wall in the soil and the forces applied to the wall in the anchor.

Below a situation with an anchored sheet pile wall in the crest of the embankment is shown. All the pictures in this paragraph have been copied from "Projectplan Waterwet Dijkversterking Gorinchem Waardenburg".

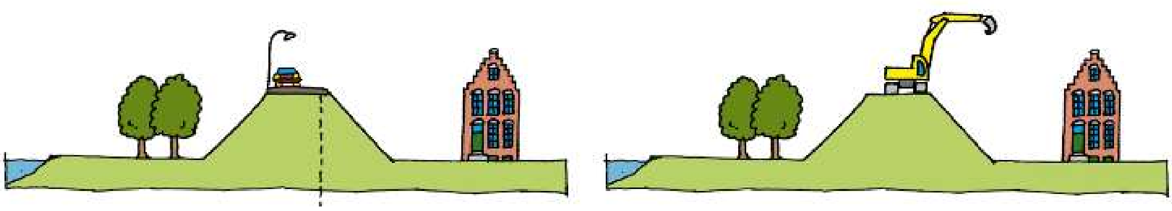

Figure 3.205 Schematic cross section of situation 1: original situation (left) and situation 2: clearing of obstacles (right)

In the first picture the original situation is shown. Due to lack of space a structural solution is selected (dotted line in the crest of the dike). As a first step the dike is cleared of obstacles (picture on the right).

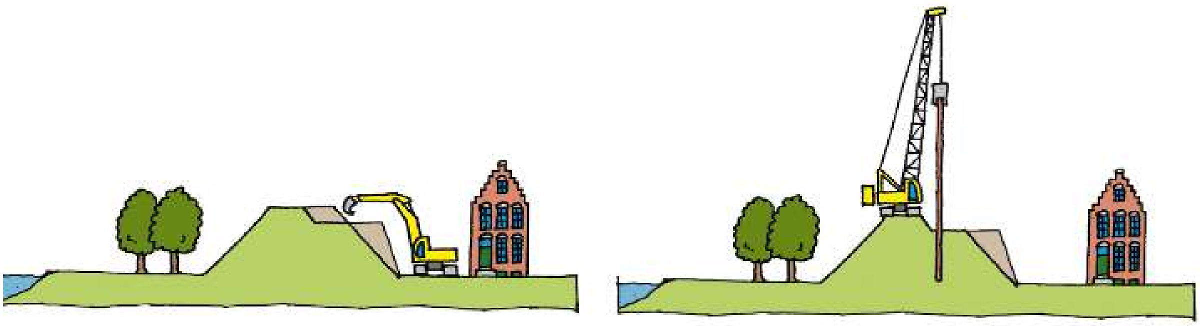

Figure 3.206 Schematic cross section of situation 3: groundworks to create a stable working environment (left) and situation 4: inserting the sheet piles (right).

Groundworks can be necessary to provide a stable working environment. After that the sheet pile wall can be inserted into the dike. The sheet pile wall has to be inserted well into the sand surface for stability. The length of the sheet pile walls vary from 15 to 20 meters.

Figure 3.207 Schematic cross section of situation 5: construction of grout anchors (left) and situation 6: fastening of anchors to sheet pile wall and add provision to diminish the influence of the sheet pile wall ground water levels (right).

To provide extra stability and limit deformations of the sheet pile wall and the soil grouted anchors are applied. Provisions are made to diminish the influence of the sheet pile wall on ground water levels.

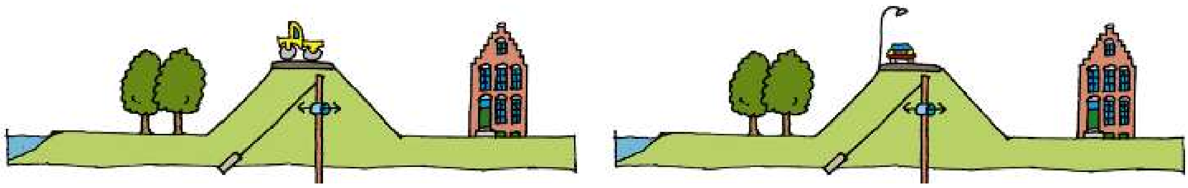

Figure 3.208 Schematic cross section of situation 7: construction of road (left) and final situation (right).

Sheet pile walls can be used at different places within the cross section of the embankment, for instance also in the toe of the inner slope of the dike. If there are intermediate sand layers it can also be a combined measure for both slope stability and backward erosion piping. In some situation, if there is an issue with crest height a sheet pile wall can be used as a retaining wall or a foundation for a retaining wall.

Applicability

The application of a sheet pile wall is usually linked with a lack of space for conventional solutions. This is usually an urban environment or next to buildings. The subsoil can consist of soft soils (peat and low density clays) and even uplifting of the blanket in the hinterland due to high pore water pressures can occur.

If the soft layers are very thick the length of the sheet piles (and thus the costs) increases, as the sheet pile wall has to be founded well into the sand layers.

The presence of a sheet pile wall can be a physical obstacle for other infrastructures, such as pipelines and cables. Also, the groundwater level on both sides of the sheet pile wall is influenced both during daily and extreme conditions. Measures can be taken to limit this influence.

Design

Design criteria

The design is primarily an ultimate limit state approach. The dike construction has to perform during peak hydraulic loads. During peak hydraulic conditions serviceability can also be an issue, as the expected deformations of sheet pile wall and dike are at their largest. If deformations are considered too large the design has to be adapted.

Which factors affect the dimensioning?

The design is complex, as the functioning of the construction is a result of the combined performance of soil, sheet pile and anchors and the interaction amongst these. Apart from this, deformation influences the performance. To get a proper design numerical (finite element) models are used, which require expert knowledge to get correct results. The design standards which are used are a combination of guidelines for constructions and numerical modelling.

In the design it can be included that the inner slope of the dike slides down partially and the hinterland is lifted by water pressure in the sand layers, if the sheet pile wall and anchoring are designed to provide the necessary stability.

How are key parameters for this technique measured or estimated?

The soil parameters are determined with conventional methods (borings, cone penetration tests and laboratory measurements). From these methods also load – deformation behaviour of soils for Finite Element calculations can be derived. Behaviour of steel sheet lies and anchors are derived from product specifications.

Other design considerations

Table 3.19 Uncertainties in applying anchored sheet pile walls

Uncertainty	*Influence on application of technique*
Increasing flood frequency, intensity or duration due to climate change.	Included in the 100 year design period
Dryer periods possibility of cracking in embankments	Not considered
Land subsidence	Included in the 100 year design period
Requirements to make embankments warn before failure or to fail slowly	Not explicitly considered, but the failure behaviour of sheet piles can limit the damage compared to soil dikes which are eroded easily
Material prices	Considered when evaluating different solutions
Environmental considerations	Not considered in design, but in choice between different solutions
Population growth/ increasing economic value in areas sensitive to flooding	Should ideally be included in the 100 year design period

Construction

Which methods of construction are available?

There are different solutions for placement of the sheet piles and getting them to the required depth, such as drilling, pushing, vibrating or liquefying. The main consideration (next to expected success of different types of placement methods) is the influence on the surroundings (vibrations, chance of damage, disturbance). The placement of anchoring elements can be a challenge near (historical) buildings.

Other construction considerations?

The availability of space can influence the choice of equipment and installation method.

Variants on the technique or construction method

Usually there is a choice between standard sheet pile profiles with standard specifications. The same holds for anchor bars with standard diameters and steel quality.

References

Design manuals/standards/codes of practice

POV Macrostabiliteit (2020). POVM Publicatie langsconstructies (PPL). Retrieved from: https://publicwiki.deltares.nl/display/HWBPMacro/Publicaties#Publicaties-POVMPublicatieLangsconstructies(PPL),2020 (in Dutch)

POV Macrostabiliteit (2020). POVM Eindige-elementenmethode. Retrieved from: https://publicwiki.deltares.nl/display/HWBPMacro/Publicaties?preview=/187991112/187991118/publicatie_POVM-Eindige-elementenmethode.pdf#Publicaties-POVMPublicatieEindigeelementenmethode(PPE),2020 (in Dutch)

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https://terinzage.gralliantie.nl (in Dutch)

https://waterschaprivierenland.nl (in Dutch)

3.2.18 Technique factsheet: Soil mix walls (slope stability)

Authors: Leen Vincke, Leen De Vos, Daniel Verastegui; Geotechnical Division of the Department of Mobility and Public Works, Flanders, Belgium

For the corresponding case study, please refer to Section 3.1.20.

Failure mechanism(s) and/or Basic function

The soil mixing method is used to improve the mechanical properties (strength and stiffness) of a natural soil. In this particular case of stabilization, the soil mixing shear walls work against slope sliding.

Description technique

The method consists of installing soil mixing wall elements perpendicularly oriented to the centerline of the river. Figure 3.209 (a) below shows and example of an unstable levee. Based on observations and back analysis a slip surface can be delineated. The mixing walls, as illustrated in Figure 3.209 (b), strengthen the weak layer along which a greater portion of the slip surface develops.

The soil mixing walls are installed down to a stronger soil layer or a few meters below the slip surface. In this way the walls become fixed at their base to stable ground. Similarly the soil mixing walls extend to some meters above the slip surface. This portion of the walls holds the unstable soil mass above the slip surface through interface friction and arching that develops between neighboring walls. The spacing between walls then becomes a relevant design factor that controls these interactions.

The soil mixing walls themselves work as shear wall elements. The shear strength of the soil mixing material directly contributes to the safety of the levee. As illustrated in figure (b) the sliding along the original slip surface is interrupted. A new smaller surface in front of the soil mixing walls may now become relevant but often not as critical as the original one. Minor stabilizing actions such as toe berms/riprap are usually enough to tackle these areas.

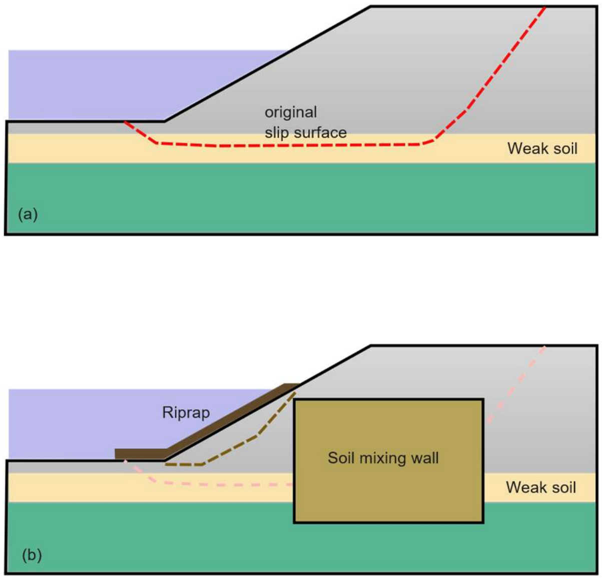

Figure 3.209 Slip surface (a) and soil mixing wall (b)

Applicability

Stabilization with soil mixing walls can be applied in situations where space constraints (urban environment) would not allow for reshaping of the slope by terracing of reducing the slope angle. The technique is virtually noise and vibration free.

It could also be a valuable alternative when the magnitude of the sliding soil mass involved is so large that other structural elements such as sheet pile walls of nails would prove insufficient. This technique could be applied to any natural levee and on a variety of soil types (sand, silts, clay, organic soils), except on rock or gravels.

Design

Design criteria

The design of this slope stabilization technique is above all an ultimate limit state approach. A well designed stabilization will eliminate the continuous sliding but some limited deformation may still take place to mobilize arching and resisting forces.

Which factors affect the dimensioning?

As a first step in the design it is necessary to estimate the initial condition of the levee and to establish the mechanism that brought it to an unstable state. A back analysis of the levee incorporating any relevant boundary conditions (e.g. river water level, ground water table, location of any visible shear fissures on the ground surface) may help providing a set of realistic shear strength parameters of the soils and estimating a sliding surface.

Once the failure mechanism is estimated, the stabilization design can be started. Essential parameters for the design include the shear strength and stiffness of the resulting soil mixing material. These parameters are difficult to assess without any specific laboratory/in situ research, as they are highly dependent not only on the mixing materials but also on the mixing quality and therefore may show significant variability.

In design guidelines of conventional soil mixing applications there are some analytical approaches available in which average parameters of the soil mixing stabilized zones can be estimated based on the parameters of the natural soil, parameters of the cemented soil and an area replacement ratio. In this way conventional stability calculations could be performed. However, depending on the configuration of natural soil layers, geometry and soil mixing wall spacing, complex interaction patterns may result. Therefore, the use of finite element models (3D) may help to provide more realistic results and to optimize the configuration of the soil mixing walls.

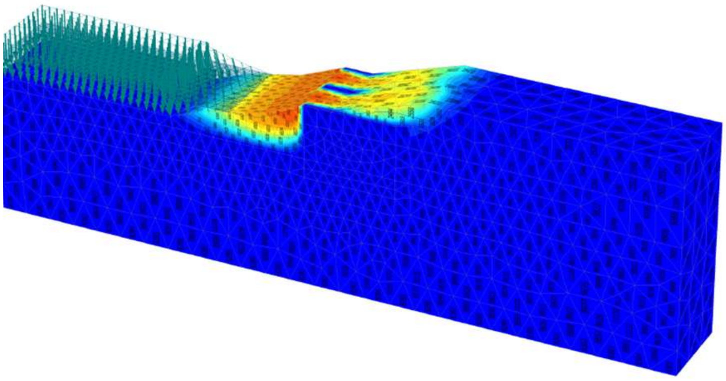

Figure 3.210 PLAXIS 3D simulation of the soil mix wall.

How are key parameters for this technique measured or estimated?

Soil strength and stiffness parameters of the natural soils can be evaluated out of a geotechnical investigation campaign, with emphasis on the stiff clay layer. Determination of critical state and residual shear angle of the stiff clay through laboratory testing is relevant.
The design of soil mixing involves first the choice of a cement type and the most appropriate water/cement ratio for mixing. While this can be investigated solely under laboratory conditions, it is advisable to install trial soil mixing wall elements and to obtain test samples from the actual wall. The reason for this is that the mechanical properties of the resulting soil mixing material are also strongly affected by the mixing procedure, water content of the natural soil, pH and salinity of the groundwater, pollution and temperature. Results of this investigation will show scatter, so a design value can be assessed based on the available data (more details are provided in the SBR CUR net manual).

Other design considerations

Construction activities on top of an unstable levee are challenging. Therefore, the design should also take the construction phase into account and propose temporary measures to render the levee safe. For example, in the case history at Melle (paragraph 1470), the height of the levee was temporarily reduced and the groundwater table was lowered through pumping during the construction activities. These measures improved the safety and minimized the sliding. Monitoring horizontal deformations at the site during construction activities was very useful as a warning sign and control tool.

Construction

Which methods of construction are available?

The soilmix process involves in-situ mechanical mixing of soil while injecting a cement- or lime-based binder using a specially designed machine. The technology is categorized based on how the binder is introduced into the soil, resulting in two main methods: the wet method (most common in Belgium) and the dry method. In the wet method a mixture of binder and water, sometimes with sand or additives, is injected and mixed with the soil. In the dry method the dry binder is directly mixed with the soil at its natural water content, then the binder reacts with the existing soil moisture, forming a soil mortar.
In Belgium and the Netherlands, only the wet method is used for the execution of soil mix walls using columns and panels. The most common systems in Belgium are CVR C-mix®, TSM and CSM.

The CVR C-mix® method uses a modified drilling rig equipped with a specially designed shaft and mixing tool. This tool rotates around a vertical axis at about 100 rpm, mechanically cutting through the soil. At the same time, a water/binder mixture is injected at low pressure (< 5 bar). The resulting soilmix elements are cylindrical columns, with diameters ranging from 0.43 to 1.03 meters. To increase productivity, two enhanced systems were developed: the CVR Twinmix® uses two mixing tools to create panels of two overlapping columns and the CVR Triple C-mix® which uses three overlapping mixing tools in a row. Figure 3.211 (left) illustrates this method.

The TSM (Turbo Soil Mixing) technique combines mechanical and hydraulic mixing principles. It features a rotating mixing tool (around a vertical axis) and uses high-pressure injection (up to 500 bar) of a water/binder mixture. The resulting soilmix elements are cylindrical columns with diameters ranging from 0.38 to 0.73m. But double and triple version are also available to produce wall elements 0.8 to 1.4 m and 1.2 to 2.1 m wide respectively.

The Cutter Soil Mixing (CSM) system, developed by Bauer Maschinen GmbH, has been mainly used for building water and earth retaining walls and other ground improvement applications. The system allows soil mixing in harder ground layers. During the process, a water/binder mixture is injected while the soil is cut, broken up, and mixed using rotating cutter wheels. These wheels rotate independently on a horizontal axis to cut through the soil. The binder mixture is injected at low pressures (usually < 5 bar). The resulting soil mix elements are rectangular panels. In Belgium, these panels are usually 2.4 meters long and 0.55 meters wide, though other sizes exist internationally.

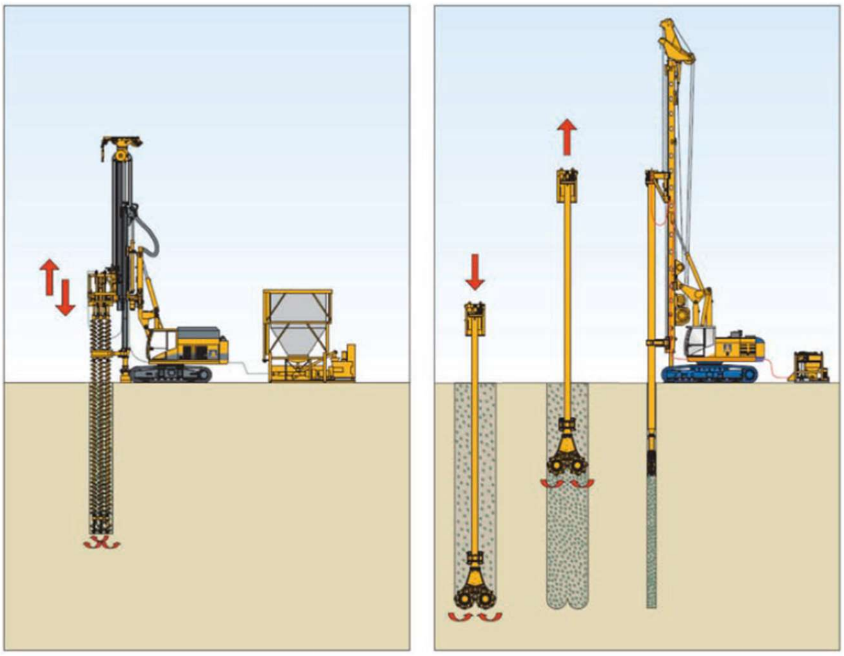

Figure 3.211 Soil mixing technique (source: Handboek Soilmix-wanden, ontwerp en uitvoering – SBR CUR net)

Other construction considerations?

When using soil mix walls, the following points should also be considered:

- Soil type: The method works on sand, silt and clay, although stiff clay may be more difficult to mix.
- Vibrations: The process of installation causes very little vibration.
- Underground obstacles: These can cause serious issues, so a thorough site investigation is important.
- Groundwater: Lowering the groundwater level is not required. However, if there is significant groundwater flow, the risk of the soil mix material washing out must be assessed.
- Nearby structures: It is possible to use soil mix walls close to existing buildings, but proper preparation is needed. This includes site investigation, possibly adjusting the size and sequence of the soil mix elements, and evaluating the impact on nearby structures.
- Chemical substances in the soil. It is recommended to investigate the presence of aggressive elements that may affect the development of cementation or that could affect the durability of the soil mixing walls.

Variants on the technique or construction method

The most common thickness of soil mixing walls in Belgium is about 0.5m. However, other thicknesses are available when necessary.

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References

Design manuals/standards/codes of practice

Soilmix Walls Manual: Design and Implementation. SBR CUR net, 2016, in Dutch.

Publications

De Vos L., De Beukelaer-Dossche M., Vincke L., Dupont E., Duyck K., Huybrechts N. & Denies N. (2019). Application of soil mixing (CSM) in stiff clay for dike stabilization. Proceedings of the XVII ECSMGE-2019, Geotechnical Engineering foundation of the future.

3.2.19 *Technique factsheet: Trench and toe drain systems (slope stability)*

This contribution is based on: *FICHE TECHNIQUE 4.4 Tranchées et recharges drainantes. Trench and toe drain system. From Recueil de méthodes et de techniques de confortement et réparation des digues de protection en remblai of the Comité Français des Barrages et Reservoirs (CFBR).*

Main authors: Y. BOUSSAFIR (Univ. Eiffel);
Contributors: M. SUTTER (INRAE) ; D. POULAIN (INRAE)
Reviewers: C. CHEVALIER (Univ. Eiffel) ; O. ARTIERES (Tencate) ; A. LE KOUBY (Univ. Eiffel) ; S. PATOUILLARD (DREAL)

Failure mechanism(s) and/or Basic function

Primary function(s):
The drainage systems have as their main function the control of hydraulic flows in and/or beneath the dike, which includes: intercepting a water level within the dike, or existing or suspected water flows through or under the dike. The water flows may be diffuse or localized. The system may also be designed to capture water flows related to animal burrows, shrinkage cracks, root conduits, etc.
Improving the hydraulic conditions through drainage also makes it possible to ensure:

- the mechanical stability of the structure, by reducing pore pressures on the slope on the protected side and by improving the cohesion and friction angle of the materials;
- stability against the risk of hydraulic uplift, by intercepting groundwater flows (Figure 3.213a and Figure 3.214).

Secondary function(s):
The drainage system also has a secondary function of filtration, by preventing the migration of soil particles thanks to the filter that is usually combined with drainage systems. In this respect, it is useful for preventing internal erosion.
The drainage system also contributes to the overall stability when it replaces a significant portion of the dike with drainage materials that have better mechanical properties than the original embankment (see Figure 3.212).

Description technique

Drainage trenches and drainage toes are very frequently implemented to manage groundwater flow within the dike. When they are positioned in the core of the dike, their main purpose is to stop internal erosion mechanisms. When they are positioned on the protected side, they allow the lowering of the water level and prevent excessive pore pressures and mechanical instabilities related to the saturation of the slope. Depending on the diagnosed needs, the placement and depth of the drainage element can intercept water flows down to the foundation level.

This sheet describes systems designed to improve dike drainage implemented without additional fill and without fundamentally modifying the geometry of the dike, such as:

- Drainage toes (Figure 3.212 and Figure 3.213)
- Drainage trenches (Figure 3.214 and 4)

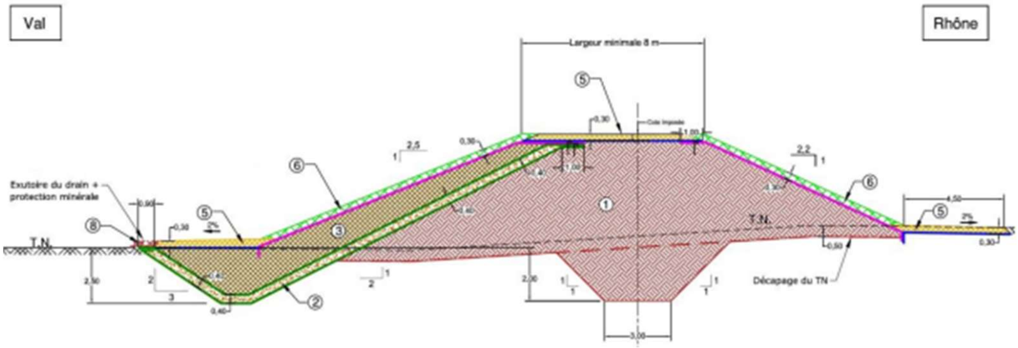

Figure 3.212 Example of a drainage layer without additional fill on the protected side (on the left of the image). In this example, the layer also drains the foundation of the dike (Source: Symadrem).

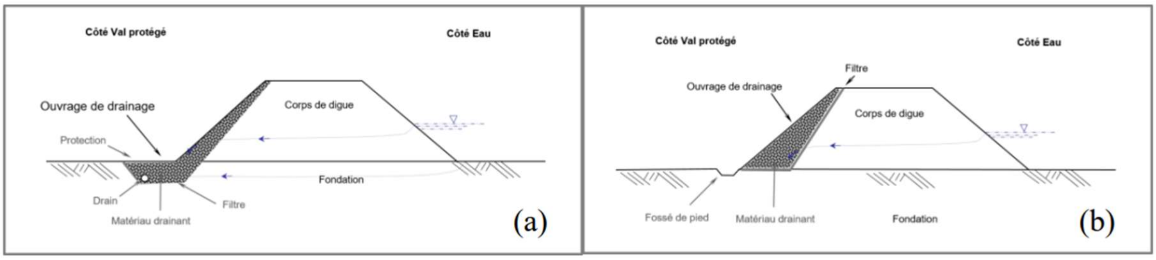

Figure 3.213 Schematic diagram of a drainage layer without additional fill on the protected side (on the left of the image), (a) with anchoring and (b) without anchoring (Source: Université Gustave Eiffel).

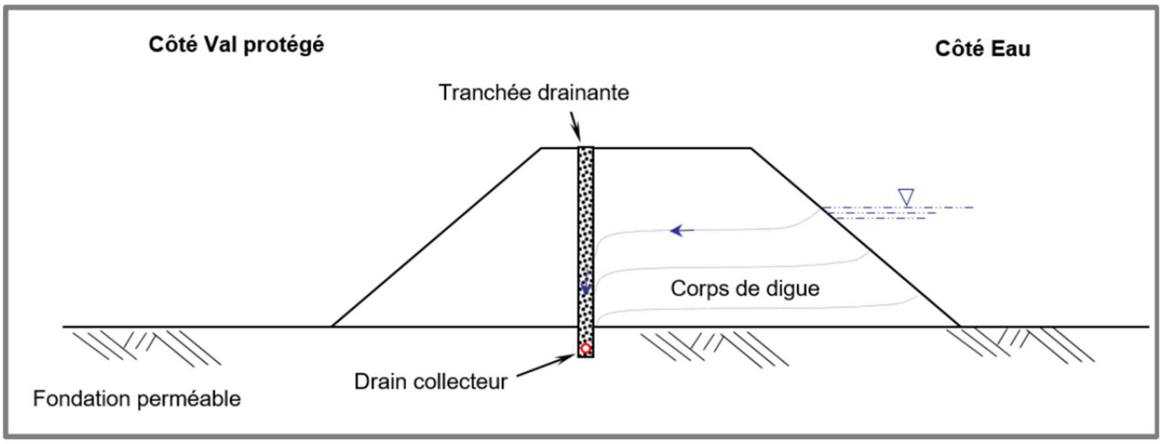

Figure 3.214 Drainage system within the body of the dike. The drainage trench intercepts internal water flows within the structure down to the foundation, thereby improving the moisture conditions of the slope on the protected side (Source: Université Gustave Eiffel)

The drainage systems are generally composed of assemblies of different materials—mainly granular materials and geosynthetics—with well-defined characteristics.
Drainage systems such as drainage blankets or vertical drains, which are more commonly used in new construction and less often in reinforcement works, will not be discussed here.

Applicability

The drainage system is designed to reduce water levels in the dike down to the level of its foundation when necessary, and to relieve pore water pressure. It must therefore be free-draining, relatively rigid, and correctly located (i.e., in contact with the water inflows
it is designed to collect). It is designed based on both a cross-section and a longitudinal profile to locate the drainage paths to the outlets.

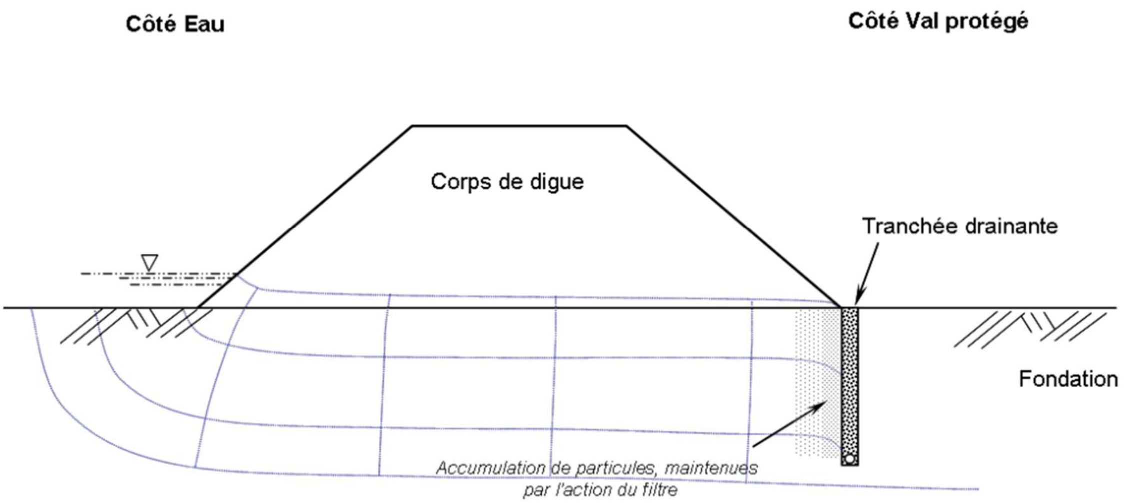

Figure 3.215 Drainage trench at the base of the slope: the system intercepts water movement

It generally includes two parts to be effective: a drainage body and a filter.

- The drainage body must allow the collection and discharge of water through gravity. It must be sufficiently permeable so that water can flow by gravity, and it must be designed to withstand hydraulic pressures.
- The filter must block soil particles at its surface while allowing water to pass through. This function is essential for maintaining the permeability properties of the drainage body. The filter must not become clogged and must be designed to allow water to pass throughout the entire service life of the structure.

Drainage systems on the protected-side slope are primarily implemented in river environments. They are installed when the objective is to stabilize a dike and when adding extra fill (widening or berm construction) is not necessary, not desired, and/or not feasible.

Design

The drainage system must be designed to evacuate free-water flow without generating pore-water overpressure, without creating a hydraulic barrier, and without deforming. Its design is primarily based on the types of soils (or materials) it will be in contact with, and on the flow rates that must be discharged. The hydraulic properties of the drainage system must be compatible with the free passage of water through the filter and within the drainage body and must allow justification of the overall stability of the structure, especially if gravity flow is desired and therefore the hydraulic head must be zero (absence of hydrostatic pressure). Standard NF G38-061 specifies the rules to be applied in these areas.

If the drainage material contributes to internal stability of the protected-side slope, the wet unit weight must be specified. This depends notably on the petrographic nature of the aggregate, as some rocks are heavier than others.

The objective is to define the following parameters:
• The location of outlets, when they exist, and any connection systems; if the system does not have natural outlets, it must be designed to store the water.
• The dimensions of the drainage system: location within the cross-section and the longitudinal profile, as well as its geometry (thickness).
• Hydraulic parameters (flow to be discharged, length to be drained).
• The specifications for the drainage materials (geosynthetics, natural materials), their specific requirements (filter criteria, unit weight, etc.), and installation conditions.

Design criteria

In addition to general design considerations, the following points must be given particular attention:

- Assessment of installation difficulty and stability conditions during construction: feasibility of excavation with or without shoring for drainage trenches (the regulatory limit for unshored excavation depth is set at 1.30 m), excavation slope angles, etc. Short-term stability conditions of the soil must be known.
- Choice of excavation technique: using conventional equipment (excavator, manual installation) or specialised equipment (trenchers, mechanized installation) depending on the required depth. The most common trench depths (and therefore achievable using a standard excavator bucket) range between 0.50 m and 2.00 m. Beyond this range, special machines are needed: trenchers can reach depths of about 12 m. In such cases, the trench is generally backfilled only with granular material, without geosynthetics.
- Assessment of installation difficulty in relation to the choice and placement of geotextile: specifying the minimum characteristics required for installation and the installation method if tensioning is expected (tensile strength, puncture resistance from angular blocks). Depending on the situation, steps (redans) may be used to integrate the drainage system into the existing structure; it must then be specified whether the geosynthetics should be anchored or not. In general, it is preferable not to anchor the geosynthetics to avoid tearing when materials are dumped over them.
- Specifications for overlap of geotextile sheets must be provided to determine quantities, for example by specifying the overlap width between sheets (often around 0.30 m).
- Installation method for the drainage material (by spot placement, requiring compaction or not, traffic direction for machinery and material delivery, etc.) and allowable drop heights must be described in the technical specifications (CCTP).
- Whether a drainage pipe (collector) needs to be added in addition to the drainage material, which may be useful in the absence of a natural outlet.
- Whether surface protection on the structure and the outlet must be included.

Construction

Which methods of construction are available?

All drainage systems are composed of:
• A drainage material (natural or geocomposite);
• A filter (natural or geotextile);
• Depending on the case, a perforated drain (pipe, collector);
• Connection systems, water collection elements, outlets, and possibly inspection chambers.

Depending on the project, the specifications expected in the CCTP will concern the choice of equipment (excavator, trencher), installation depth, wall stability (need for shoring), and phreatic levels (water table elevation). Figure 3.216 illustrates a construction site involving the installation of a drainage layer.

Figure 3.216 Construction of a drainage layer at the toe of the slope on the protected side. The excavation shape was created using an excavator bucket. In the foreground, the installation of a geosynthetic filter; in the background, the drainage gravel placed on the geotextile, awaiting coverage by the filter and a layer of topsoil – Amboise (37). The excavation depth does not require shoring, and the slopes are stable in the short term (Photo credit: DREAL Centre Val de Loire).

Figure 3.217 Drainage berm on a Loire dike: the granular part stands out clearly from the body of the dike. The ditch located between the toe of the slope and the maintenance track serves as the collection system for drainage water. There is no specific surface protection on this system, as the manager wishes to keep it visible during periods of operation (Photo credit: DREAL Centre Val de Loire).

Figure 3.218 Construction of a drainage trench at the toe of the dike on the protected side. Note: Safety instructions for personnel working on site are specific to each country; in this case, the Samira Dike in Niger (sheet C3-6). (Photo credit: Olivier Artières)

Other construction considerations?

Work progress rate:
Several tens to a few hundred linear meters per day, depending on installation depth, type of reinforcement used, and the complexity of the site.

Treatment of singular points (ends, included structures in particular):

- Care must be taken regarding specific difficulties related to managing interfaces, ensuring the hydraulic continuity of both the drainage system and the filter, and adapting the technique around any crossing structure.
- Avoid creating voids due to over-excavation during installation of the components (mainly the filter). When placing geotextiles vertically, they must be held in place against the trench walls.
- Ensure continuity of the filter throughout the entire system (overlapping of strips or stitching) and at the junctions of singular elements (outlet, inspection chambers, crossing structures).

- Crossing structures require particular attention
- The search for a natural outlet — meaning a low point in the topography capable of receiving the drainage water — can be challenging. Gravity drainage through a hydraulic gradient toward a natural outlet may be impossible. This can occur when the drainage system lies significantly below ground level. In such cases, the drainage structure must provide temporary storage capacity. For reference, the discharge rates to be evacuated are generally low, on the order of 10 to 1000 liters per hour per 100 linear meters (depending on the permeability of the surrounding soil) and are discontinuous over time, with demand concentrated during periods when the system is under hydraulic load.

Variants on the technique or construction method

Possible variants of drainage systems without additional fill are based on the choice of the system (for example, geocomposites or mixed solutions), as well as its layout and geometry. This may lead to selecting a toe-of-dike drainage system such as a drainage layer or a drainage trench. In some cases, toe drainage can be combined with foundation drainage in order to avoid potential uplift pressures.

The drainage structure may be reduced to a simple interface — as in the case of a drainage trench — when hydraulic gradients are not too high, or conversely may be oversized to the point of becoming a part of the embankment zoning, also providing part of the dike's stability due to its own weight. This type of variant may require a full reconsideration of the layout of the various structural components within the dike, verification of phreatic lines, internal stability checks, installation techniques, and even the phasing of general earthworks.

Although uncommon in France (but more frequent on systems such as those along the Mississippi River), relief wells can serve as a variant of a toe drainage trench. Drainage is achieved discontinuously through a series of circular wells located at the toe on the protected side. The principle relies on gravity-driven lowering of water levels between the wells.

Other variants may be based on the choice of materials used to build the drainage structure. Variations may suggest natural materials with different grain size distributions or other properties (such as angularity) compared to standard commercial products. This can lead to proposals involving sands, crushed gravel, rounded gravel, or small rockfill. Turnkey solutions are often proposed that use geosynthetic or geocomposite materials, or combinations of natural materials with geosynthetics or geocomposites, which frequently lead to a final solution involving drainage combined with additional fill.

The use of alternative materials in drainage structures (such as slag, shredded tires, demolition debris, etc.) is generally not recommended, because their continuous contact with water makes the environment highly vulnerable to pollution risks associated with the material.

In general, the use of natural sand filters tends to be limited to high-risk structures such as dams. Installation and supply constraints along long stretches of dike also influence the final choices. Added to this is the fact that local natural filter material resources are often not compliant with the filter criteria requirements.

If the stability of the structure on the protected side is a concern, it is possible to install drainage underneath newly added fill, which may affect the required footprint of the works.

An alternative to drainage is to block groundwater flow using vertical cut-off systems, impermeable facings, or watertight fill placed on the water side. It should be noted that new innovative techniques are emerging in the field of waterproofing and can serve as alternatives in certain situations, such as resin injections, biocalcification, etc.

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Guide remblayage des tranchées et réfection des chaussées de 1994. LCPC-Sétra : ce document permet la validation des matériels de compactage adaptés à de faibles surfaces et de petits volumes, pour des objectifs de compactage de remblai (densification q4 et q3)

Guide drainage routier de 2006. Sétra : on trouvera dans ce document des références concernant les prescriptions en matière de matériaux drainants, les formes et structures drainantes les plus couramment mises en œuvre et les éléments de contrôle qualité à exiger.

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Appendix A: Survey Results

A.1 List of questions

YOUR ROLE
What is your current role, from which perspective do you work on levees? (i.e. consultant, scientific researcher, levee manager, national or local authority, ...)

COUNTRY
On which country (or countries) is your experience based?

ENVIRONMENT
On which type of environment is your experience based? (multiple answers are possible)

FAILURE MECHANISMS
Which failure mechanisms or failure paths are your primary concern with regards to retrofitting and reinforcement of levees? If multiple mechanisms are relevant please indicate ...

ADDITIONAL COMMENTS
Do you have any further comments with regards to the previous question?

CRITERIA FOR DESIGN
Please indicate the importance of the criteria below for you design. Rating: 0 is not relevant, 3 is extremely important.

- Increasing flood frequency, intensity or duration due to climate change.
- Dryer periods possibility of cracking in levees
- Land subsidence
- Requirements to make levees warn before failure or to fail slowly
- Multifunctional levees
- Material prices
- Environmental consideration
- Population growth/ increasing economic value in areas sensitive to flooding

ADDITIONAL COMMENTS
Do you have any further comments with regards to the previous question?

UNCERTAINTIES FOR THE LONG TERM DESIGN
What are your most important uncertainties if you want to design for 100 years? Rating: 0 is not relevant, 3 is extremely important.

- Increasing flood frequency, intensity or duration due to climate change.
- Dryer periods possibility of cracking in levees
- Land subsidence
- Requirements to make levees warn before failure or to fail slowly
- Multifunctional levees
- Material prices
- Environmental consideration
- Population growth/ increasing economic value in areas sensitive to flooding

ADDITIONAL COMMENTS
Do you have any further comments with regards to the previous question?

RETROFITTING TECHNIQUES
Are your retrofitting and reinforcement needs met by the available measures/techniques and guidelines/publications?

CHOICE OF TECHNIQUES
Please indicate which techniques you use to meet your needs.

DESIRED TECHNIQUES
Please indicate which types of techniques you are missing.

CASES OF REINFORCEMENT OR RETROFITTING
Are you aware of, or involved in, reinforcement or retrofitting of levees using innovative techniques.

CONTACT
Can we contact you for more information? If so, please fill in your email address below.

FINAL COMMENTS
Do you have any other comments or suggestions in relation to this survey or the initiative?

A.2 Responses

Table appendix A.1 Number of respondents

		Mentions
Number of respondents		*68*

YOUR ROLE
What is your current role, from which perspective do you work on levees? (i.e. consultant, scientific researcher, levee manager, national or local authority, ...)

Table appendix A.2 Responses to survey question on experience/role

Role	***Mentions***
Consultant	*29*
Scientific researcher	19
Levee manager	3
National authority	16
Local authority	5
Contractor or supplier	2

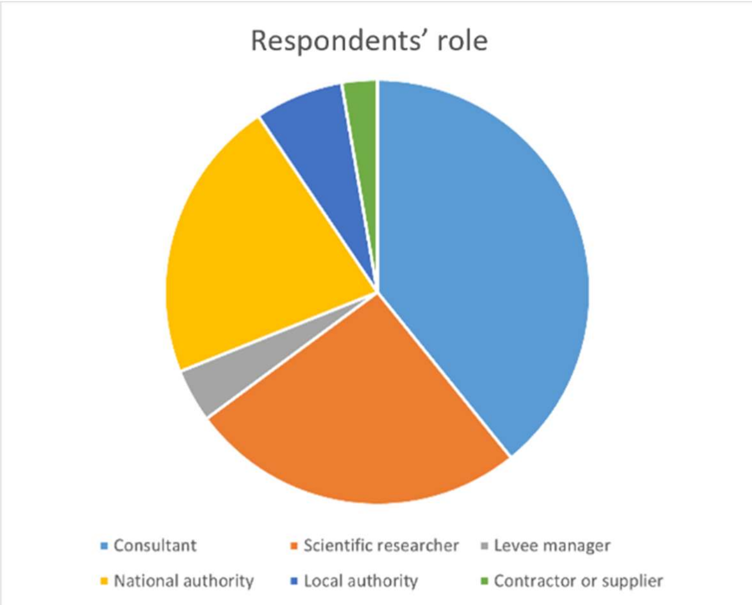

Figure appendix A.1 Overview of respondents' roles

COUNTRY
On which country (or countries) is your experience based?

Table appendix A.3 Responses to survey question on experience/country

Country	***Mentions***
France	10
Mexico	9
Netherlands	7
Indonesia	4
Germany	4
Japan	2
Italy	2
UK	2
Poland	2
Peru	2
Hungary	2
Romania	2
Slovenia	2
Canada	2
Argentina	1
Belgium	1

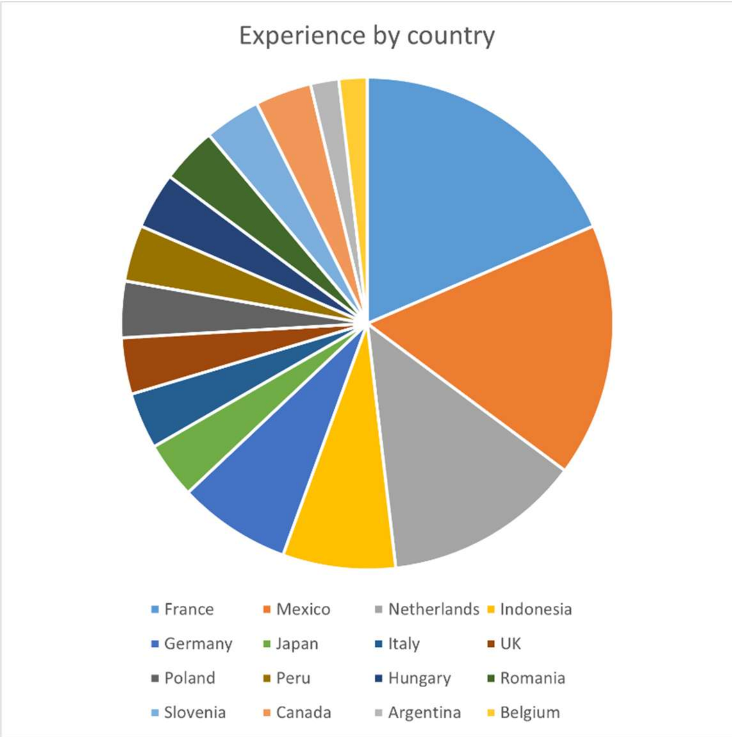

Figure appendix A.2 Overview of experience by country

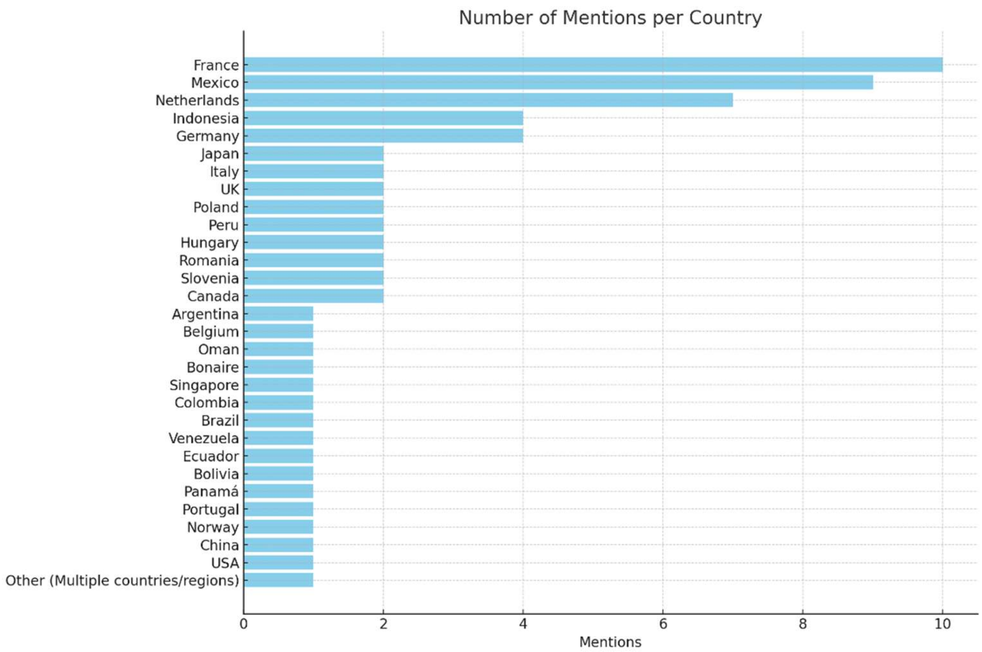

Figure appendix A.3 Overview of experience by country

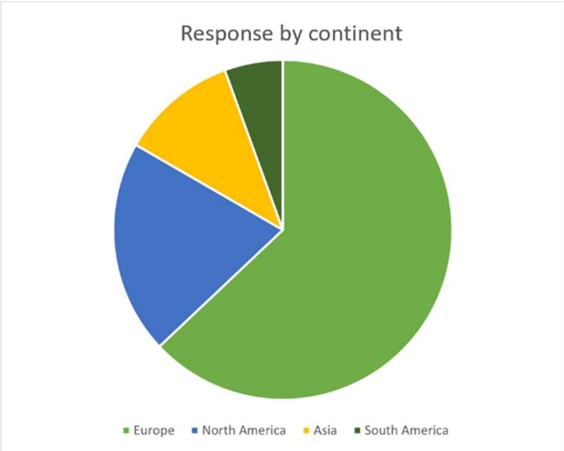

Figure appendix A.4 Overview of response by continent

ENVIRONMENT
On which type of environment is your experience based? (multiple answers are possible)

Table appendix A.4 Responses to survey question on experience/type of environment

Type of environment	***Mentions***
River levees	*60*
Coastal levees	31
Estuarine levees	18
Mountain stream levees	12
Mine levees	1
Small dam	1
Dams	2
Tailings dams	1
Slope stabilisation	1
Dams of all kinds	1
HPP dikes	1
Secondary levees	1

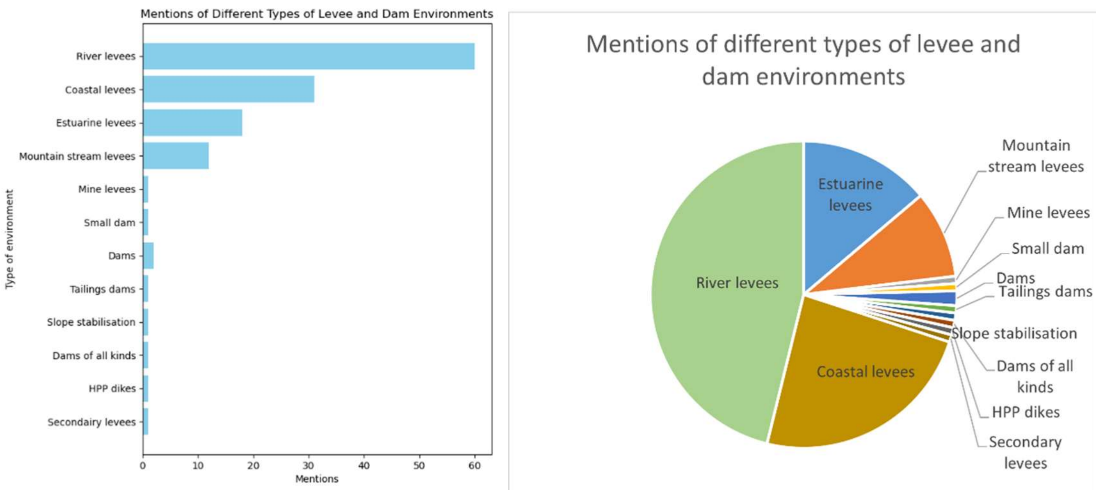

Figure appendix A.5 Overview of experiments/types of environments

FAILURE MECHANISMS

Which failure mechanisms or failure paths are your primary concern with regards to retrofitting and reinforcement of levees? If multiple mechanisms are relevant please indicate only the most important four.

Table appendix A.5 Responses to survey question on relevant failure mechanisms

Failure Mechanism	***Count***
Internal erosion	51
Slope sliding	42
External erosion on the water side	40
Failure due to animal behaviour or vegetation	31
Liquefaction and breach flow sliding (in under water slopes)	22
External erosion on the land side	16
Failure due to objects on or in the levee (i.e. house, windmill pipelines,...)	15
overtopping	2
Piping	1
Failure of dikes reinforced with structural elements	1
Geotechnical instability, backwards erosion piping	1
Subsoil failure (sand boil)	1
seismic conditions and failure of foundation	1
new dimensioning heights	1
Lack of maintenance	1
Overtopping and inability to operate closures in the levee system	1

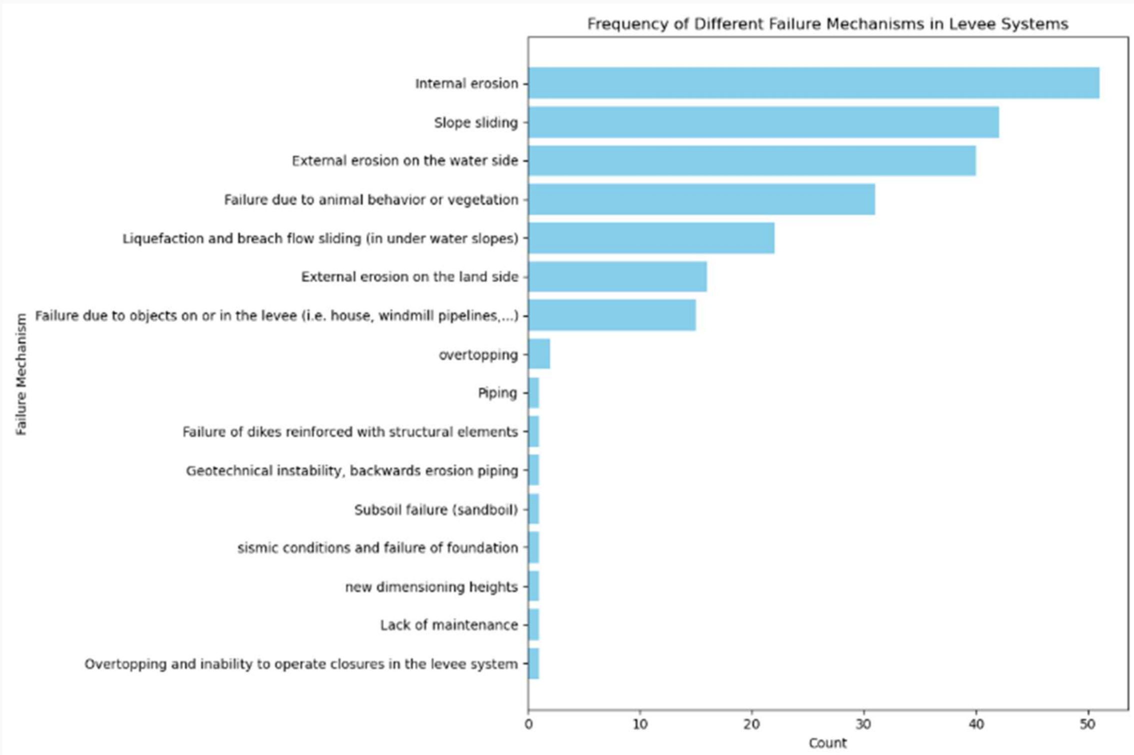

Figure appendix A.6 Overview of relevant failure mechanisms

ADDITIONAL COMMENTS
Do you have any further comments with regards to the previous question?

- Piping is also sometimes occurring
- All mechanisms are important, also their interaction during a failure scenario (or failure path). Slope sliding is probably the less important but can anyway be the consequence of other mechanisms. Erosion by overtopping is different as overtopping can be avoided (spillways).
- Overtopping is one of the most frequent risk to manage (not mentioned in the previous question).
- Priority will vary between coastal and riverine levees due to loading variation.
- Overtopping
- This is a general answer, it depends highly on the location and geometry.
- The relevance of external erosion on the land side is due to boars.
- I'm familiar with the entire Dutch practice of evaluation and design of dikes, which I now export
- Based on our experiences in our region, the most problematic point of heightening of the dikes is the hydraulic load on the subsoil (dH).
- Overflow erosion and scouring, deterioration due to rainfall
- Tailings dams exhibit all of these failure mechanisms.
- SE debe considerar también el efecto del sismo, condiciones de saturación, modificación geométrica del talud, sistemas de protección inadecuados, y las propiedades geotécnicas cambiantes (The effect of the earthquake, saturation conditions, geometric modification of the slope, inadequate protection systems, and changing geotechnical properties must also be considered.)
- External erosion on the land side concerns clay embankments whereas slope sliding concerns coarse-grained embankments
- In some cases it is possible to combine causes such external erosion with damage by animals
- Most of the levees were designed and built some 50-60 years ago. Therefore, design standards are obsolete, materials and structures are degraded, thus creating conditions to trigger failure mechanisms.
- Vieillesse des ouvrages, Manque d'entretien pendant des décennies qui ont conduit à la dégradation des ouvrages (Old age of the structures, Lack of maintenance over decades which led to the deterioration of the structures)
- The U.S. levee portfolio evaluation shows that 40% of levees are likely to breach from overtopping, 17% are likely to breach due to internal erosion, 15% are likely to breach due to external erosion (waterside), and 9% are likely to breach due to inability to operate a closure in the levee system.
- Depends on the region we work. In the Netherlands above is primary concern; overseas liquefaction due to seismic loading is our primary concern

CRITERIA FOR DESIGN

Please indicate the importance of the criteria below for your design. Rating: 0 is not relevant, 3 is extremely important.

Table appendix A.6 Responses to survey question on relevant uncertainties current design . (0: not relevant, 3: extremely relevant)

Criteria	***0***	***1***	***2***	***3***
Increasing flood frequency, intensity or duration due to climate change.	3	10	14	39
Dryer periods possibility of cracking in levees	9	23	21	12
Land subsidence	9	31	14	9
Requirements to make levees warn before failure or to fail slowly	10	20	19	17
Multifunctional levees	9	17	26	12
Material prices	6	24	23	12
Environmental consideration	1	14	27	23
Population growth/ increasing economic value in areas sensitive to flooding	6	15	22	22

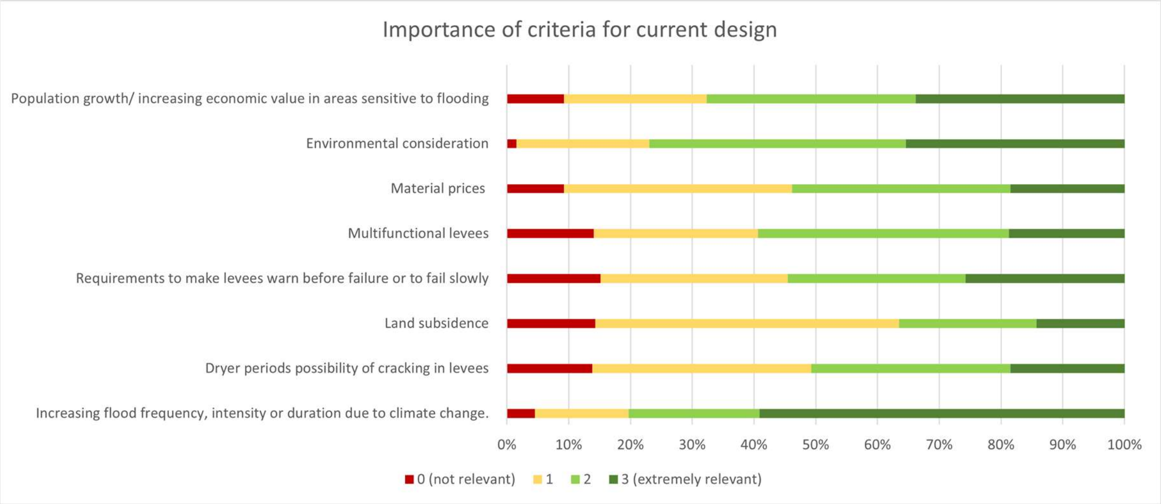

Figure appendix A.7 Overview of criteria and their importance for current design

ADDITIONAL COMMENTS
Do you have any further comments with regards to the previous question?

- My answer is general and abstract. Each project can have specific criteria in a very different order.
- Missing social acceptance
- Climate change is of importance, water levels will rise, the question is only when. Multifunction vs primary function of flood defence is a problem
- The change in our water safety system is the primary reason for reinforcements of our dikes
- Of course, environmental aspects are of relevance for the construction but shouldn't dominate the technical measures
- Unfortunately the dike development projects or even a structure reconstruction became significantly expensive so they can only be financed by dedicated governmental decision. Not the highest priority nowadays..
- River water level waveform controlling failure modes
- Monitoring of these structures and in-depth characterization of the dike materials should be considered.
- El tipo de suelo, si es cohesivo, friccionante o combinación de ambos, variabilidad de condiciones piezométricas que afectan el estado de esfuerzos efectivos, estado de drenes e instrumentación a largo plazo (The type of soil, whether it is cohesive, frictional or a combination of both, variability of piezometric conditions that affect the state of effective stresses, state of drains and long-term instrumentation)
- Levees behaviour is different in humid and semi-arid regions
- local conditions, quantity and quality of materials are of major concern when designing a levee
- Wave impact is of major interest for coastal levee
- In Netherlands we see the growing importance of sustainable improvements (in use of material as well in adaptability)

UNCERTAINTIES FOR THE LONG TERM DESIGN

What are your most important uncertainties if you want to design for 100 years? Rating: 0 is not relevant, 3 is extremely important.

Table appendix A.7 Responses to survey question on relevant uncertainties for long term design for 100 years. (0: not relevant, 3: extremely relevant)

Criteria	**0**	**1**	**2**	**3**
Increasing flood frequency, intensity or duration due to climate change.	1	3	15	47
Dryer periods possibility of cracking in levees	5	20	25	15
Land subsidence	12	22	20	9
Requirements to make levees warn before failure or to fail slowly	11	17	21	12
Multifunctional levees	6	22	21	13
Material prices	9	27	17	8
Environmental consideration	4	13	22	25
Population growth/ increasing economic value in areas sensitive to flooding	4	15	20	26

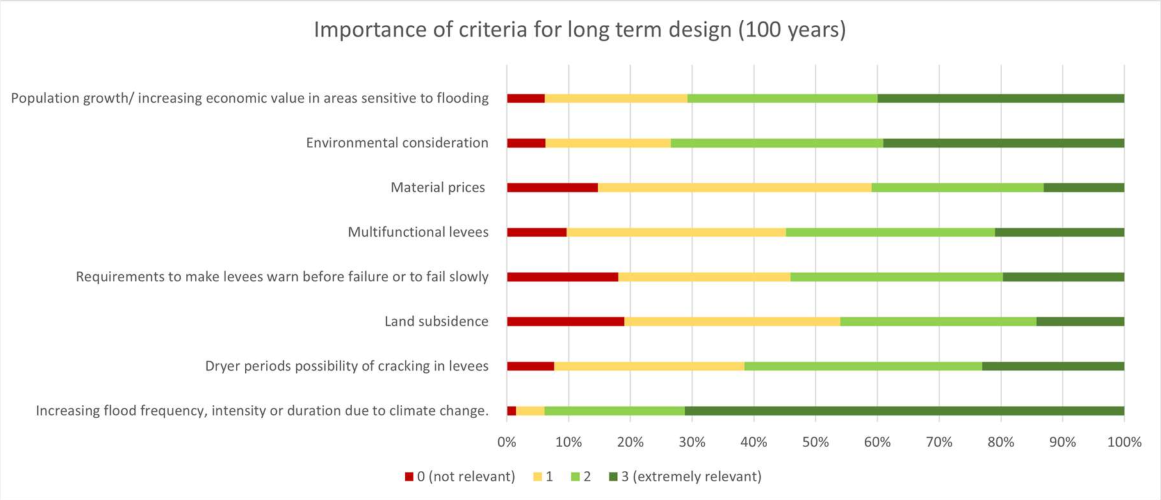

Figure appendix A.8 Overview of criteria and their importance for long term design (100 years)

ADDITIONAL COMMENTS

Do you have any further comments with regards to the previous question?

- The area of a flood defence is subject too many regulations and stakeholders (more urbanization) which make planning very uncertain and projects costly.
- Some regions I work in have land subsidence rates of up to 20 cm/year. That means the entire system is uncertain...
- The most uncertainties are the subsoil conditions due to the "exhausted" layers bellow the defence system.
- Post-earthquake behaviour in terms of deformations must be considered for long-term analysis.

- variabilidad de las propiedades del suelo, modificación de las parámetros mecánicos, aspectos del cambio de periodo de vibración, asentamiento por consolidación primaria y secundari (variability of soil properties, modification of mechanical parameters, aspects of vibration period change, settlement by primary and secondary consolidation)
- Depends on each project

RETROFITTING TECHNIQUES
Are your retrofitting and reinforcement needs met by the available measures/techniques and guidelines/publications?

Table appendix A.8 Responses to survey question on needs met by guidelines/publications

Response	***Count***
No	12
Yes	23
Partially	31

CHOICE OF TECHNIQUES
Please indicate which techniques you use to meet your needs.

There were many responses to this question. They are grouped below in 1) mentioned techniques 2) measurement 3) referral to guidelines.

Mentioned techniques:

- Grouting
- soilmix panels for slope sliding
- cemented soils
- Always compaction, In Poland we always install impermeable element in body of the new or modernized levee and in subsoil only if it is necessary (too permeable, too large velocities, too large uplift, etc.) Impermeable elements use in Poland in body are geomembrane or Bentonite geotextile in the waterside slope of the levee or cut off walls (sheet piles or Trencher or Deep soil mixing) from the crest
- Structural reinforcements
- Sheet piles (https://doi.org/10.4233/uuid:78df5e2b-740e-4268-a821-ed0ccaae93e5)
- Rock armour, geotextiles, occasionally concrete
- Cut off walls in levees, banks on landside.
- George tile and sheet piles
- drainage system, seepage barrier, revetments
- We usually do vertical trench shields but recently we applied jet grouting also in a 700 m section.
- Stress-strain analysis, monitoring and observation, new construction and stabilization methods
- Levee toe drain method
- Soil improvement, soil reinforcement, use of filters and drains, berms, impermeable barriers, among others.
- anclas, modificación geométrica, colocación de drenes horizontals
- Biotic and abiotic protection of the landside slope subjected to overflow-induced surface erosion

- find a solution to the problem that arises, soil improvement, wall reinforcement, etc.
- Gabion wall + renomatress installation at the surface, plus increased bearing capacity at the base.
- Reconstructions d'ouvrages en remblais, en matériaux étanches (argile A1...), épaissisement d'ouvrages, Battage de palplanches, pose de géomembranes étanches
- anti-burrowing solutions to prevent dyke failures
- Armouring with concrete, blocks, rocks and geotextiles
- techniques for preventing the spread of animals and plants with deep root systems, early damage detection systems, design to prevent processes such as seepage or desiccation, restoration of the ground layer and the use of additional sealing techniques
- Techniques for reinforcing and repairing flood protection levees; Techniques for controlling overflow (weirs, lowering the crest of levees) ; no construction of new levees
- Structural modifications/improvements, Material Replacement
- Additional berm, slope impermeabilization with geomembranes, sheet piles

Measurements

- temperature measurements to see in which ground layer piping occurs,...

Guidelines

- Key for our reinforcements are our legal assessment and design framework(BOI), in which most knowledge is presented.
- For standard techniques like reinforcements with clay there is a lot of information available in guidelines and publications. For innovative measures, like for example geotextiles to prevent piping, the availability of useful references is limited.
- Reliability based design methods
- Literature review, hydraulic model tests
- Numerical analysis and limit equilibrium analysis.
- Use of different manual and regulation
- No guidelines available in Canada
- Dutch guidelines, with standard procedures

DESIRED TECHNIQUES
Please indicate which types of techniques you are missing.

There were 9 responses to this question:

- soil mixing in levees
- Design for transitions, in particular but not only crossing pipes.
- Liquefaction guidelines are still quite straightforward and depend highly on the geometry and less on the physical processes.
- Geotextiles
- Use of geosynthetics
- Techniques for making levee sticky and ductile.
- National guidelines
- modern techniques and methods for retrofitting and reinforcement
- Reinforcement using treated soil

CASES OF REINFORCEMENT OR RETROFITTING
Are you aware of, or involved in, reinforcement or retrofitting of levees using innovative techniques.

Table appendix A.9 Responses to survey question on cases of reinforcement or retrofitting

Response	***Count***
No	31
Yes	35

CONTACT
Can we contact you for more information? If so, please fill in your email address below.

The responses of this question are not included in this report.

FINAL COMMENTS
Do you have any other comments or suggestions in relation to this survey or the initiative?

- Green construction
- Optical fibre sensors for levee monitoring
- In France, we don't need to considerate climate change to conclude that most of (old) French levees need to be refurbished. Project are always designed considering extreme flood.
- Environmental consideration and social acceptance are more and more important.
- I believe that there is a need of more careful geotechnical characterization and analysis of river levee stability (using statistical approaches, whenever possible), to avoid overconservative design on interventions, that are often very costly.
- In our region the subsoil is mainly gravel-type with high permeability covered with a thin (semi-)permeable topmost layer. Due to the extremely high floods in the last 2 decades numerous events we recorded and we believe that there shall be already dangerous "pipes" underneath the dikes. So older soil investigations (1980's) may not be valid anymore or not enough detailed to locate the problematic points.
- SE debe ampliar el tema de la exploración mínima para conocer el suelo, involucrando aspectos topográficos, exploración geofísica, cambios del estado de esfuerzos, variabilidad de la presión de poro, instrumentación geotécnica, casos históricos de éxtito y casos de ingeniería forense. (The topic of minimum exploration to understand the soil should be expanded, involving topographic aspects, geophysical exploration, stress state changes, pore pressure variability, geotechnical instrumentation, historical success stories, and forensic engineering cases.)
- International Manuals on good practices for repair, retrofit, reinforcement, upgrade of levees could be useful.
- The need for the future is to have adaptable reinforcement or retrofitting, allowing multiple rehabilitations stages over time according to the evolutions of the environment.
- Nous mettons en œuvre des travaux de renforcement mais avec des techniques qui sont usuelles et pas forcément innovantes. Aujourd'hui ce n'est pas tant la technique qui freine les projets mais plutôt la règlementation, toujours plus lourde... (We implement reinforcement work, but with techniques that are common and not necessarily innovative. Today, it's not so much the technology that's holding back projects, but rather the increasingly cumbersome regulations...)
- A guideline to include measures to prevent failures in dykes needed. If such a publication is planned within the TC 201 WG, I would be glad to contribute to the works.