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DOCTORAL THESIS

Transport Infrastructure Asset Management

A holistic framework for transport infrastructure asset management applied for inland waterways

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DISSERTATION

Asset Management von Verkehrsinfrastrukturen

Ein ganzheitlicher methodischer Rahmen für das Asset Management von Verkehrsinfrastrukturen mit praktischer Anwendung für Binnenwasserstraßen

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von

Dipl. -Ing. Katrin Haselbauer Bsc.
Matr. Nr.: 0425815
Operngasse 26/11; A-1040 Wien

Gutachter:

Univ. Prof. Dipl.-Ing. Dr. techn. Ronald Blab
Institute of Transportation

Research Center of Road Engineering
Vienna University of Technology

Ao. Univ. Prof. Dipl.-Ing. Dr. techn. Norbert Krouzecky
Institute of Hydraulic Engineering and Water Resources Management
Research Center of Hydraulic Engineering
Vienna University of Technology

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Kurzfassung

In der europäischen Transportpolitik wird Binnenwasserstraßen schon länger eine hohe Bedeutung zugemessen, da sie im Vergleich zu anderen Verkehrsträgern einen deutlich geringeren Energieverbrauch und niedrigere Emissionen aufweisen und zudem geringere externe Kosten verursachen. Trotz freier Transportkapazitäten und eines äußerst geringen Bedarfs an Infrastrukturinvestitionen hat das Ausbleiben notwendiger Investitionen zu einer geringen Verfügbarkeit und Zuverlässigkeit sowie Mängel an Effizienz geführt, weshalb das Transportvolumen auf der Wasserstraße seit einigen Jahren stagniert. Im Gegensatz zu konkurrierenden Verkehrsträgeren wie Straße und Bahn haben zudem bislang methodische Ansätze für eine systemische Sichtweise des Verkehrsträgers gefehlt, die ein Management und eine Optimierung des Gesamtsystems Wasserstraße auf Basis von Lebenszyklusansätzen erlauben. Neben fehlenden Definitionen für eine durchgängige Zielqualität der Infrastruktur und national organisierten Verwaltungsapparaten limitiert auch die fallbezogene Maßnahmenplanung die Effizienz von Erhaltung und Entwicklung bzw. dem Verkehrsträger generell.

Die Zielsetzung dieser Arbeit war es daher, einen neuartigen Asset Management Ansatz basierend auf bestehenden ausführlich dargestellten Methoden im Asset Management spezifisch für die Wasserstraße zu entwickeln. Die erstmalige Beschreibung der Infrastrukturqualität von Wasserstraßen auf Basis eines Verfügbarkeitsmodells bildet den methodischen Kern der Arbeit. Ein zentraler Bereich der Dissertation konzentriert sich daher auf die Entwicklung, mathematische Formulierung, Darstellung und Verifikation eines umfassenden Verfügbarkeitskriteriums. Der methodische Ansatz zum Asset Management von Binnenwasserstraßen repräsentiert die Sicht des Betreibers und beinhaltet eine konkrete Darstellung von Kosten, Wirkung und umweltschonender Umsetzung aller wesentlichen Maßnahmen im Hinblick auf diese Infrastrukturqualität anhand der Parameter Verfügbarkeit und Zuverlässigkeit. Ebenfalls im Ansatz enthalten sind die Auswirkungen dieser Infrastrukturqualität auf die Nutzer (Transportkosten), wodurch alle Investitionen durchgängig sowohl für Betreiber alleine, als auch für Betreiber und Nutzer gemeinsam optimiert werden können. Anhand dieses Verfügbarkeitskriteriums wird gezeigt, dass eine Optimierung von Maßnahmen an einzelnen Stellen nur begrenzt wirtschaftlich effizient sein kann, wenn diese nicht in Hinblick auf eine durchgängige Verfügbarkeit einer gesamten Transportroute hin optimiert werden. Weiters zeigen die Ergebnisse der praktischen Umsetzung, dass die Optimierung auf eine durchgängig vorhandene, ausreichende Fahrrinnentiefe entscheidend ist, da in einem seriellen System eine einzelne Seichtstelle die Abladetiefe und damit die Auslastung der Flotte maßgebend limitiert.

Die im Rahmen der Dissertation und einem Pilotprojekt für den österreichischen Abschnitt der Donau entwickelten Ansätze wurden bereits in einem Softwaretool mit dem Namen WAMS (Wasserstraßenmanagementsystem) umgesetzt. Dieses WAMS erlaubt die Analyse der Infrastrukturqualität und Planung von durchgängigen Erhaltungsmaßnahmen auf der gesamten österreichischen Donau und wird bereits im laufenden Betrieb der Wasserstraßenverwaltungs- Ges.m.b.H VIADONAU erfolgreich eingesetzt. Die Analyse der empirischen Daten aus den durchgeführten Zustandserfassungen und umgesetzten Maßnahmen der letzten fünf Jahre zeigen eine klare Übereinstimmung des methodischen Ansatzes in Hinblick auf Nutzer- & Betreiberkosten sowie die Infrastrukturqualität. Die Resultate der Forschung belegen weiters, dass eine alleinige Umsetzung des entwickelten Ansatzes in Österreich aufgrund der langen Transportwege zu kurz greift, um die Wasserstraße dauerhaft wettbewerbsfähig gegenüber anderen Verkehrsträgern zu halten. Diese Forschungsergebnisse sind bereits in den politischen Entscheidungsprozess eingeflossen und haben einen langsamen Umdenkprozess in der Strukturierung und Finanzierung der Wasserstraße bewirkt. So wurde gemäß Beschluss der Donau-Verkehrsminister von 2014 ein „Fairway Rehabilitation and Maintenance Masterplan“ (Juli 2015) beschlossen, aus dem ein substantieller Wille zur Verbesserung von Durchgängigkeit und Verfügbarkeit durch entsprechende Investitionen ablesbar ist.

Abstract

Due to their low energy consumption, low emissions and low external costs compared to other modes of transport, inland waterways are highly regarded in European transport policy. Despite large free transport capacities and very low infrastructure investments needs of this natural mode of transport, inland waterways in Europe generally face the problem of low availability, reliability and effectiveness of investments, leading to a stagnation of transport volumes. In addition, there are no methodological approaches available allowing for systematic holistic management and optimization of waterways based on life cycle costs in contrast to other competing modes of transport such as road or rail. Furthermore, because of the lack of common definitions of a uniform target infrastructure quality and a decision patchwork of responsible waterway administrations, a case-by-case approach to measure planning, among other aspects, are a limiting factor for efficient maintenance and development of inland waterways.

One major objective of this thesis is to develop a new specific asset management approach for inland waterways based on common existing approaches, which are both described in detail in the methodology chapter. The novel description of necessary infrastructure quality, based on a model of fairway availability, forms the methodical core of this thesis. Thus, a central part focuses on the development, mathematical formulation, illustration and verification of a comprehensive availability criterion. The developed methodological approach to asset management of inland waterways represents the view of infrastructure operators including costs and impact duration for all relevant measures in terms of infrastructure quality. The impact of all measures on the waterway is defined by their impact on the performance parameters availability and reliability. Furthermore, the developed approach ensures environmentally friendly planning and implementation of measures based on a comprehensive analysis of measure impact. Moreover, all investments may be optimized from the perspective of a waterway operator, as well as from the operators and users, based on a comprehensive transport cost model that is linked to the availability approach. Based on the developed approach, it is possible to prove that an economically efficient optimization of measures cannot be accomplished on single sections alone. Due to the serial characteristic of waterways, it is instead necessary to aim at uniform and continuous fairway availability on the entire transport route. In addition, the results of the practical verification underline the importance of an optimization towards a uniform, continuously available and sufficient target fairway depth, as one single shallow section can limit the loaded draught of passing vessels and thus the utilization of the entire vessel fleet.

The presented methodical approaches have been developed within the scope of this thesis, research project and pilot application for the Austrian stretch of the Danube. Furthermore, these approaches are already implemented in a software tool called WAMS (Waterway Asset Management System). This tool allows for an evaluation of infrastructure quality and planning of maintenance measures on waterways. On the basis of a successful testing phase with data from already conducted riverbed surveys and implemented measures of more than five years the methodical approach has been validated. Currently, the WAMS – Software is successfully being used for availability analysis and measure planning by VIADONAU, which is the leading international waterway operator in the Danube region. However, the results of the research also prove that an implementation on the Austrian stretch of the Danube falls short due to rather long average transport distances. Instead, concerted actions on the entire Danube will be necessary in order to be competitive on the transport market in the future. These research results are already incorporated into the decision-making process, indicating a change regarding management and financing of the Danube waterway. Within the framework of a resolution from December 2014, the transport ministers of the riparian Danube countries have agreed on a “fairway rehabilitation and maintenance masterplan”, reflecting their substantial intention to invest in improving continuous availability and reliability of the Danube waterway.

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1 INTRODUCTION

1.1 Relevance of infrastructure asset management and inland waterways

In Europe, the infrastructure of the transportation of persons and goods is well developed and was constructed years or even decades ago for most modes of transport. The preservation and the development of this infrastructure are essential for the European economic system and the welfare of European citizens. With an increasing age of the infrastructure system, loads as traffic congestion, climate conditions are leading to structural damage, which may result in system failure with severe consequences for infrastructure operators and customers as a worst case scenario, if appropriate measures are not implemented in time.

Operators of transport infrastructure therefore have to implement operational, maintenance and rehabilitation measures in order to improve infrastructure condition and counteract asset value reduction. However, applying both the most effective and efficient measure for complex infrastructure systems requires a systematic approach to asset conservation. Infrastructure asset management systems provide this systematic approach for infrastructure operators and cover all main tasks of responsibility. Such systems have gained considerable importance in recent years.

Asset management systems provide an overview of the existing infrastructure asset portfolio, the condition of infrastructure assets and allow an optimization of measure selection as well as the optimization of measure programs, such that within an available budget the best possible infrastructure condition can be provided to customers and the asset value may be secured in the long term as well. Furthermore, in times of scarce budgetary resources, the documentation of investment decisions together with the bases for decisions is becoming increasingly important for public infrastructure operators.

Additionally, in most European countries not only transport modes are in a competition with each other in terms of public funding, which is because also other public infrastructures such as schools or hospitals have to be financed as well. To be able to decide where these limited budget resources are used most efficiently (i.e. provide the greatest benefit in terms of macro economy), relevant performance indicators of these infrastructures have to be known. These include infrastructure costs, transport capacity as well as emissions, such as noise and carbon dioxide. Nevertheless, the decision of budget allocation remains an administrative and political issue.

In order to allow reasonable decisions, it is therefore necessary to know all advantages and disadvantages of transport modes as well as their life cycle costs. Such an assessment is possible based on asset management approaches. These systems have a long tradition for road and rail systems, but not for inland waterways. Especially in times when emissions have to be reduced, an increase of barely-utilized inland waterways is seen as favorable. In turn, an appropriate approach is required to allow an assessment of efficiency and affordability compared to other modes of transport.

Market research provided evidence that no suitable approach for inland waterways is available. This was taken as an opportunity to develop a new asset management approach fulfilling the requirements of inland waterways based on known principles and components of common asset management systems.

1.2 Problem definition and objectives

Although inland waterways are highly regarded in European transport policy, they have to face niche existence as a mode of transport. Regardless of very low infrastructure investments costs as a near natural transport mode, low external costs, low energy consumption and low emissions, inland waterways in Europe generally face the problem of a administrative patchwork, low availability and effectiveness of investments leading to a rather declining importance compared to other modes of transport. Contradictory to the desired development with an increasing share of inland navigation on the transport market, the current development trends show a gradually declining importance compared to road and rail. As inland waterways are a linear mode of transport which allows no detours, a single shallow section with inadequate fairway depths does not allow passing and in turn leads to a closure of inland navigation. For this transport system with a serial arrangement of shallow sections, the section with the lowest fairway depth is decisive for the utilization of the entire fleet. This means that, for main transport routes, uniform fairway depths must be available throughout the year on the entire transport route. For navigation companies the availability of minimum fairway widths and depths is crucial both for planning individual transport trips and for being competitive throughout the year. There are a number of shortcomings and challenges for inland waterways that still persist today and hinder necessary investments in inland waterway infrastructure despite considerable efforts:

- Very dynamic condition development of the fairway that can only partially be controlled (floods) leading to lower predictability and reliability
- Limited catchment area – due to fixed pre- and end haulage costs
- Favorable framework conditions mainly for bulky goods (logistical structure)
- Comparatively long duration of transport and limited scheduling of transport conditions
- Lower external costs of inland navigation are not included in the market prices
- No guarantee for the availability of agreed fairway parameters (e.g. European Agreement on Main Inland Waterways of International Importance AGN, Danube Commission Recommendations DC)
- Limited utilization of existing fairway parameters due to uncertainties
- Over-aged waterway infrastructures and inland vessel fleet
- Administrative and logistical barriers as hindrance for trading of goods
- Limited profitability of individual investments in waterways due to differences in infrastructure maintenance policies and approaches in neighboring countries
- Limited political commitment in some riparian countries to invest in waterways
- Pressure from environmental groups to limit river maintenance and engineering activity

Without considerable efforts in mitigating the above mentioned shortcomings and challenges, the medium to long-term outlook regarding modal share is poor as well. In order to obtain uniform condition parameters (fairway depth) for the entire transport infrastructure, it has to be considered as inevitable to develop a comprehensive asset management approach which accounts for essential characteristics of inland waterways, subsequently followed by a Danube-wide implementation. Therefore, an approach capable of providing comprehensive management tasks was developed on behalf of the Austrian water authority VIADONAU within a pilot project for the Austrian stretch of the Danube, including a software-technical implementation as waterway asset management system also termed as WAMS.

1.3 Scope and system boundaries

The Danube as European inland waterway currently faces a slowly declining importance compared to other modes of transport. In comparison to other modes of transport, cargo transport on inland waterways will only be competitive, if sufficient fairway parameters are provided on entire transport routes. Currently there are a number of international agreements and recommendations setting ambitious targets for common minimum fairway parameters (width, depth, days per year). However, due to political, technical, environmental and economic reasons, necessary fairway maintenance and river engineering works on the Danube are not implemented, resulting in unsatisfactory fairway conditions compared to these targets. To overcome these shortcomings a new waterway asset management approach aiming at an increased availability of fairway width and depth in days per year was developed. This new holistic multidisciplinary approach for the development, maintenance, rehabilitation and replacement of waterway assets is based on a comprehensive life cycle costing approach. Based on periodic riverbed surveys, current water levels and discharge the impact of maintenance and river engineering works on the availability of fairway widths and depths can be modeled allowing a calculation of real time availability for the transport industry. Innovative alert systems based on an empirically derived backfilling behavior of critical bottlenecks after measures allow for a determination of timing and efficient measure implementation. On an entire transport route only a continuous increase of available fairway loading depths may lead to an efficient allocation of investments leading to a decrease in transport costs that may be considered as a benefit of implemented measures. Results of the research indicate that only concerted actions on a transnational level of all waterway authorities and stakeholders under a common strategy will lead to efficient investments. The implementation of such a harmonized common strategy together with an implementation of necessary maintenance and river engineering works will be crucial for inland navigation and can be described as an overarching goal for the future of the Danube as competitive mode of transport in the heart of Europe. The presented waterway asset management system WAMS takes a first step towards this goal and is currently being implemented on the Austrian stretch of the river Danube. The thesis provides an overview of the methods for a calculation of fairway availability together with first results for an actual section of the Danube in Austria. Furthermore, possible measures to improve availability are introduced together with an in-depth description of implementation and optimization of dredging measures. Additionally, an algorithm for an optimization of all measures, not only for individual sections but the entire Danube, is given. This algorithm provides an optimization for constrained budgets or recommended fairway parameters as well as total costs both for waterway measure and transport costs.

For this thesis, it is furthermore important to clarify that the core tasks of waterway management, namely the identification and illustration of fairway conditions, forecast of condition development, economic comparisons and the derivation of performance indicators, have to be distinguished from other tasks and research areas, such as planning and detailed analysis river engineering measures, detailed analysis of sedimentation processes and hydrodynamic-numeric-simulations (HN-simulations). Since these research areas pursue highly different objectives, they are in no competition with waterway asset management and will never be replaced by any asset management approach. On the contrary, these collaborations will facilitate new evaluation options for all research partners. The presented doctoral thesis is to be seen as a classic interdisciplinary cross-section work, linking fundamental system components of various research fields. As a result an economic assessment of measures becomes feasible for the first time along with an automatic derivation of important performance indicators. However, due to the wide scope of this cross-section work each necessary module cannot cover its respective fields in its entire academic depth.

2 TRANSPORT INFRASTRUCTURE ASSET MANAGEMENT

2.1 Introduction to infrastructure

Several examples in history indicate that *success* and *progress* of *human society* strongly depend on *available physical infrastructure* for distributing resources and essential services to the public. The quality and efficiency of this infrastructure affect both the quality of life and the continuity of economic and business activity. Historical development of economic and social systems closely parallels *phases* of *infrastructure development* and *urban growth*. Demands on infrastructure and related services increase as people expect a higher standard of living and public services [Uddin, W. 2013]. An increasing functional division of labor and economy of scale with subsequent efficiency gains is only possible with highly functional multimodal goods transport. *Economic development* is therefore closely linked to the *demand of transport* and the *supply of transport infrastructure.* Several famous statements underline the importance of infrastructure systems. Thus, a report from the US National Science Foundation on civil infrastructure systems [NSF 1994] states:

> *"A civilization's rise and fall is linked to its ability to feed and shelter its people and defend itself. These capabilities depend on infrastructure – the underlying, often hidden foundation of a society's wealth and quality of life. A society that neglects its infrastructure loses the ability to transport people and food, provide clean air and water, control disease, and conduct commerce."*

An analysis of the historical development of transport infrastructure [Grübler, A. 1990] shows that the expansion of the extent of network lengths of transport modes containing phases of growth, saturation and decline by substitution may be described by typical growth and saturation models (Figure 1). Common models of evolutionary economics include diffusion effects[1] of technological advancement and the substitution process among different transport modes (e.g. canals, railways, roads and airways) and were investigated in 1990 for the first time by Grübler based on an analysis of historical materials from several developed countries.

According to current theories in evolutionary economics, the development of human economic activities is similar to the process of biological evolution. In transport industry development, the evolution of several transport modes (canal, railway, road and air transport) is associated with the substitution of main energy sources, such as animal force, coal and petroleum. These transport modes go through their respectively life cycles consisting of birth, growth, saturation and declination stages, gaining and losing a leading position. At the beginning, new modes and their infrastructure are complementary, becoming gradually independent, and eventually take the place of competing modes. The culminating point of the saturation stage for every growth curve is restricted by natural environment and resource conditions of a society, for example the required water transport condition, the corresponding energy supply, the ground area for road building and parking and the territorial sky for flight. The decline for each transport mode is decided by aging of mode assets, reinvestment needs and the competition pressure imposed by new emerging transport technologies [Rong, Ch. 1999]. Several major research studies indicate that development and expansion of infrastructure networks generally follow the long waves of cyclical developments in economic growth (Figure 1).

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[1] *Diffusion models attempt to describe growth and saturation processes and assume that time series are approaching towards saturations limits. Diffusion models differ from life cycle models as they do not depict degeneration.*

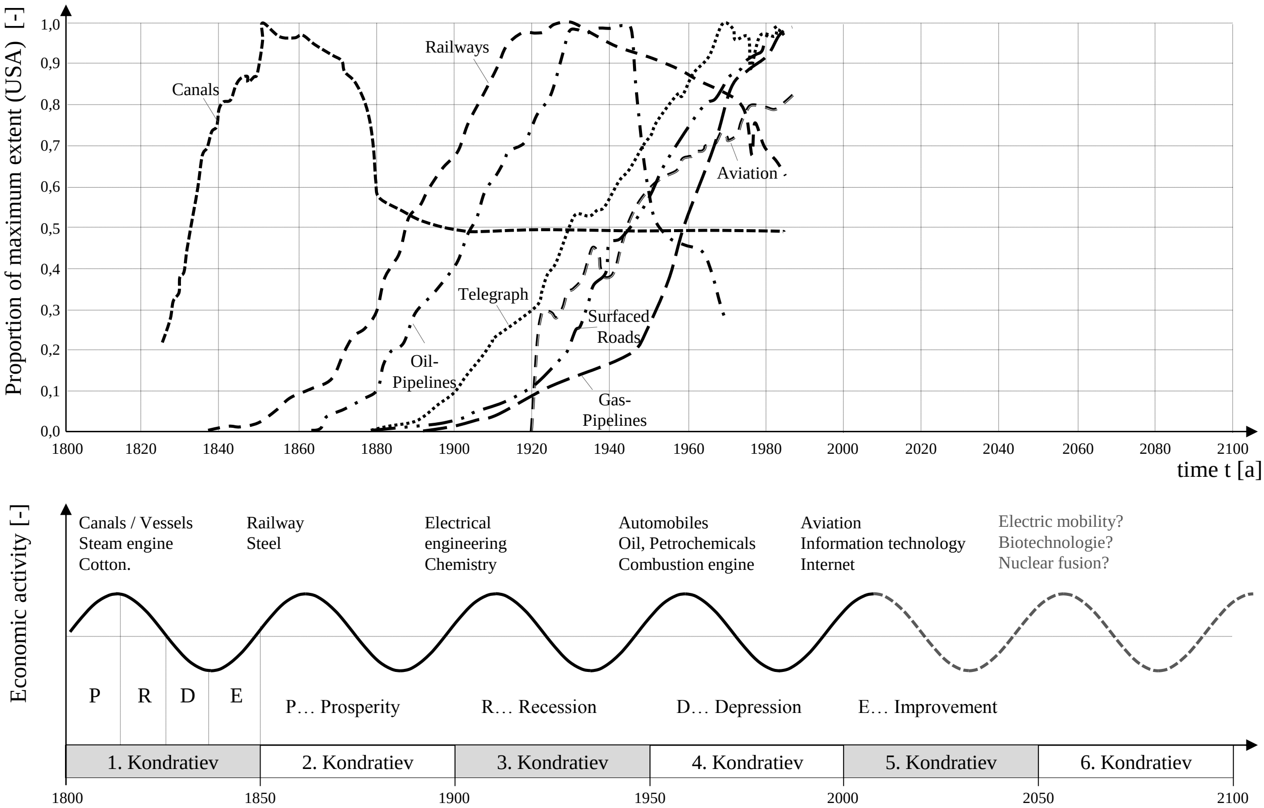

Figure 1: Transport network development from 1800 - 2000 in the United States as proportion of maximum extent [Grübler, A. 1990] and [Hoffmann, M. et al. 2014] compared to economic growth cycles according to the Kondratiev theory [Nefiodow, L. 2014]

In line with Schumpeter's theory of long cycles[2], *innovation* in *evolutionary economics* is considered as a *main source* to break the *equilibrium* and prompts *economic growth* and *economic structural changes.* Fluctuations in innovation cause fluctuation in investment and those cause cycles in economics growth [Rosenberg, N. 1994]. According to Kondratiev, the period of a wave of economic activity may be described by a cycle of alternating intervals between high growth and intervals of relatively slow growth ranging from forty to sixty years. In general, a wave may be divided into the four phases: *prosperity*, *recession*, *depression* and *improvement*. Every wave of innovation lasts approximately until the margins between investments and revenues from the new innovation or sector fall to the level or below alternative sectors. In this situation a new technology (e.g. infrastructure), which originally increased a capacity to utilize new sources from nature, reached its limits and it is not possible to overcome this limit without an application of another new technology (or infrastructure). Typical growth cycles in the world economy over time include periods like steam engine and canals, railways and steel, electrical engineering and chemistry, petrochemicals and automobiles and finally information technology (Figure 1) [Nefiodow, L. 2014].

A more detailed analyzes of historical development of transport modes indicates that an increasing division of labor requires *sufficiently large markets* leading subsequently to a demand for *increased transport volumes* and *speeds* that are reflected in requirements for transport infrastructure and realized transport volumes. Therefore, *system speed*, *capacity*, *availability* and *reliability of transport systems* in particular played a major role in growth and decline of infrastructure systems.

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[2] *Innovations are seen clustering around certain points in time periods (neighborhoods of equilibrium), when entrepreneurs perceive that risk and returns warrant innovative commitments. These clusters are assumed to lead to long cycles by generation periods of acceleration an aggregate growth [Rosenberg, N. 1994].*

Assuming fixed costs of transport infrastructures, an *efficient operation* becomes feasible if *large settlements are connected*. In the past, the development of many historical cities was directly linked to the *availability of waterways*. Thus, inland navigation on waterways developed rapidly between the years 1800 and 1850 and faced a long decline parallel to the rise of railway infrastructure from 1840 to 1920. The decline of railway infrastructure development fell together with the rise of paved roads and road transport from 1890 to 2000. For long distance passenger transport, airport infrastructure showed huge increases starting from 1960 until now. For long distance goods transport, maritime shipping soared during the same time period. In accordance with the infrastructure development cycle described above, *new*, more *reliable* and *faster infrastructures* are *permanently expanded* while existing infrastructures, which gained their importance have to be *maintained or disposed*.

The realization of new transport infrastructure is very costly and thus a huge burden for tight budgets of regions or states, even with a booming economy. If these investments have already been made, then the necessary reinvestment needs are at first very low but later steeply increase in the form of reinvestment waves. Refinancing of infrastructure assets at the end of their service life represents a far more significant challenge. Whether or not these reinvestment needs from aging (transport) infrastructure are met depends on the economic situation and is a necessary prerequisite for future economic development. While in developing countries new structures are built first and foremost, in countries with well-developed infrastructure the task of maintaining existing infrastructure efficiently becomes more and more important. In both cases financial viability is crucial for sustainable availability of the transport infrastructure. The growing scarcity of available budgets generally represents an increasing challenge for infrastructure operators. In the public sector, budgetary constraints are a result of increasing competition between different areas of expenditure (such as school infrastructures, transport infrastructures and hospitals) on the one hand, and increasing competition between different modes of transport on the other hand. For private infrastructure operators, budget shortages for individual tasks are mainly a result of increasing competition between business areas. These budget shortages can be considered as one of the main reasons for the need of higher investment efficiency that have led to an increasing demand for asset management and life cycle cost approaches.

With increasing age of infrastructures, system failures become more frequent leading to a significant increase of subsequent total costs for infrastructure operators, consisting of downtime costs, accident costs and restoration costs and more. As a result, the issue of improving manageability of failures becomes more important as well as an optimized and transparent decision making process. Additionally, increasing customer requirements regarding transport safety, which are already reflected in a stricter legislation, are considered as contrary to the economical use of scarce public resources and therefore require the derivation of appropriate condition boundaries. Infrastructure owners may pursue various paths in dealing with their facilities. The fundamental decision therefore falls between sustainable maintenance of the affected infrastructure and the associated development of asset management approaches on the one hand, and gaining profit from the exploitation of the existing structure by doing nothing on the other hand.

The complexity and size of infrastructure networks enhanced the development of various infrastructure asset management systems. By implementing these approaches in software tools, a retransmission of these systems to a majority of infrastructure operators becomes possible. Thus, an efficient use of resources and preservation of asset value as well as a transparency of decisions becomes feasible providing a good overview on infrastructure condition as well.

2.2 Terminology and definitions in asset management

Asset management systems are a management systems focusing on *maximizing* the *effectiveness of assets* (all types of fixed assets) as well as the *efficiency of necessary operational services*. The focus of asset management thus primarily aims at the management of enterprises which consider machinery, plant engineering and maintenance as a strategic success factor. Assets used for *value creation* are analyzed in detail in terms of *potential risks* in different life cycle phases. Since this term led to a number of confusions due to various definitions and specifications in different sectors, like portfolio management, IT and industry, as well as different translations for various Anglo-American areas, standardization within ISO 55000 was first published in 2014 by the British Standards Institution (BSI). Based on this standard selected important terms and their further use in the thesis are explained in the current chapter:

1) **Infrastructure**

   Infrastructure[3] is the basic *physical* and *organizational structure* needed for the operation of a society or enterprise, or the services and facilities necessary for a functioning economy. Infrastructure can be generally defined as a *set* of *interconnected structural elements* that provide a framework supporting an entire structure of development. It is an important term for judging a country or region's development. Viewed functionally, infrastructure facilitates the production of *goods* and *services*, and also the *distribution of finished products to markets*, as well as basic social services such as schools and hospitals; for example, roads enable the transport of raw materials to a factory. In military parlance, the term refers to the buildings and permanent installations necessary for the support, redeployment, and operation of military forces. Research by anthropologists and geographers shows the social importance and multiple ways that infrastructures shape human society and vice versa. Infrastructure consisting of physical systems can be owned and managed by either public agencies or private enterprises, or by both. In this thesis the word infrastructure refers to physical systems or facilities aiming at providing transportation as an essential public service.

2) **Organization**

   A person or group of people that has its own functions with responsibilities, authorities and relationships to achieve its objectives. The concept of organizations includes, but is not limited to, sole-trader, company, corporation, firm, enterprise, authority, partnership, charity or institution, or part or combination thereof, whether incorporated or not, public or private. [The British Standards Institution, 2014]. All organizations[4] have a management structure that determines relationships between the different activities and the members, and subdivides and assigns roles, responsibilities, and authority to carry out different tasks. Organizations are open systems - they affect and are affected by their environment.

3) **Management**

   In terms of infrastructure management, management describes the coordination and judicious use of means and tools, such as funding and economic analysis to optimize output or accomplish a goal of infrastructure operation [Uddin, W. 2013]. Management is goal-directed towards some purpose.

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[3] *http://en.wikipedia.org/wiki/Infrastructure; [Last access: 20.02.2015]*
[4] *http://www.businessdictionary.com/definition/organization.html; [20.02. 2015]*

4) **Asset**

An asset is an item, thing or entity that has potential or actual value to an organization. The value will vary between different organizations and their *stakeholders*, and can be *tangible* or *intangible*, *financial* or *non-financial*. Physical assets usually refer to equipment, inventory and properties owned by the organization. Physical assets are the opposite of intangible assets, which are non-physical assets such as leases, brands, digital assets, use rights licenses, intellectual property rights, reputation or agreements. [The British Standards Institution 2014]. Infrastructure assets are physical facilities or integral components of a particular physical system that is constructed and maintained to serve public needs [Uddin, W. 2013].

5) **Portfolio**[5]

In the context of financial management, portfolio is described as grouping of (financial) assets such as stocks, bonds and cash equivalents, as well as their mutual, exchange-traded and closed-fund counterparts. Portfolios may be held by individual investors or managed by (financial) institutions (or hedge funds). A portfolio is designed according to the investor's risk tolerance, time frame and investment objectives. The monetary value of each asset may influence the risk/reward ratio of the portfolio and is referred to the asset allocation of the portfolio. A broader definition of portfolio also includes physical assets (compare asset portfolio).

6) **Asset Portfolio**

Asset portfolios include the assets that are within the scope of the asset management system. An asset portfolio is typically established and assigned for managerial control purposes. An asset management system can encompass multiple asset portfolios. Portfolios for physical hardware might be defined by category (e.g. plant, equipment, tools, and land). Software portfolios might be defined by software publisher, or by platform (e.g. PC, server, mainframe). [The British Standards Institution, 2014]

7) **Portfolio Management**

Portfolio management[6] describes the art and science of making decisions about investment mix and policy, matching investments to objectives, asset allocation for individuals and institutions, and balancing risk against performance.

8) **System**

The word system has been appropriated for many purposes, such as circulatory system, drainage system, and highway system. Dictionaries say that a system is a regularly interacting or interdependent group of items comprising a unified whole [Uddin, W. 2013].

9) **Asset Management**

Asset management is defined as *"the systematic and coordinated activities and practices through which an organization optimally and sustainably manages its assets and asset systems, their associated performance, risks and expenditures over their life cycles for the purpose of achieving its organizational strategic plan".* Asset management involves the balancing of costs, opportunities and risks against the desired performance of assets, to achieve organizational objectives. The balancing might need to be considered over different timeframes. Asset management does not focus on the asset itself, but on the value that the asset can provide to the organization. The value (which can be tangible or

[5] *http://www.investopedia.com/terms/p/portfolio.asp; [Last access: 20.02.2015]*
[6] *http://www.investopedia.com/terms/p/portfoliomanagement.asp; [Last access: 20.02.2015]*

intangible, financial or non-financial) will be determined by the organization and its stakeholders, in accordance to the organizational objectives. In asset management, organizational objectives are translated into technical and financial decisions, plans and activities [The British Standards Institution, 2014].

Asset management describes as a business process and a decision making framework that covers an extended time horizon, draws from economics as well as engineering, and considers a broad range of assets. The asset management approach incorporates the economic assessment of trade-offs among alternative investment options and uses this information to make cost-effective investment decisions.

10) **Infrastructure Asset Management**

Infrastructure asset management includes systematic, coordinated planning and programming of investments or expenditure, design, construction, maintenance, operation and in-service evaluation of physical infrastructures and associated facilities. These activities range from initial information acquisition to the planning, programming and execution of new construction, maintenance, rehabilitation, and renovation; from the details of individual project design and construction to periodic in-service monitoring and evaluation, and financial management [Uddin, W. 2013]. Advanced asset management allows providing a sustained level of service defined by the customers at the lowest life cycle costs.

11) **Infrastructure Asset Management System**

The framework of an infrastructure asset management system consists of methods, procedures, data, software, policies, decisions, budgets and funds, etc. that link and enable the carrying out of all the activities involved in infrastructure asset management.

ISO 55000 defines asset management systems as a set of interrelated and interacting elements of an organization, whose function is to establish the asset management policy and asset management objectives, and the processes needed to achieve those objectives. In this context, the elements of the asset management system should be viewed as a set of tools, including policies, plans, business processes and information systems, which are integrated to give assurance that the asset management activities will be delivered. Although asset management requires accurate asset information, asset management system means more than a management information system [The British Standards Institution, 2014].

12) **Asset Management Excellence**

Asset management excellence is the balance of performance, risk, and cost to achieve an optimal solution [Campbell, J. D. et al. 2010]. An ideal infrastructure asset management system would coordinate and enable the execution of all activities so that optimum use is made of the funds available while maximizing the performance and preservation of infrastructure assets and provision of services. It would serve all management levels in the organization and would be structured to be adaptable to its entire infrastructure [Uddin, W. 2013].

13) **Asset Management Plan**

Relates to a consistent description for implementing the asset management strategy and delivering the asset management objectives - based on specified activities and use of resources, assignment of responsibilities and timeframe [The Institute of Asset Management, 2014].

14) **Asset Management Strategy**
Long-term optimized approach to management of the assets, derived from, and consistent with, the organizational strategic plan and the asset management policy [The Institute of Asset Management, 2014].

15) **Asset Management Policy**
Principles and mandated requirements derived from, and consistent with, the organizational strategic plan, providing a framework for the development and implementation of the asset management strategy and the setting of the asset management objectives [The Institute of Asset Management, 2014].

Figure 2 provides an overview of the context and relationship between the above introduced key terms in asset management. At its core is the asset portfolio containing all essential assets, *corresponding to the objectives* of asset management.

The assets managed within the portfolio may be *contiguous* (e.g. infrastructure networks) or *independently* (financial assets) requiring subsequently different optimization approaches. Coordinating many facets of asset management requires a system of direction and control, normally a software system, ensuring that the right asset information gets to the right people. This asset management system is used by the organization to direct, coordinate and control asset management activities for all assets within the portfolio. It can provide improved risk control, assuring that the asset management objectives will be achieved on a consistent basis.

But not all asset management activities can be formalized using an asset management system. For example, aspects such as leadership, culture, motivation, behavior, which have a significant influence on the overall achievements, may be managed by the organization using approaches outside the asset management system [The British Standards Institution, 2014].

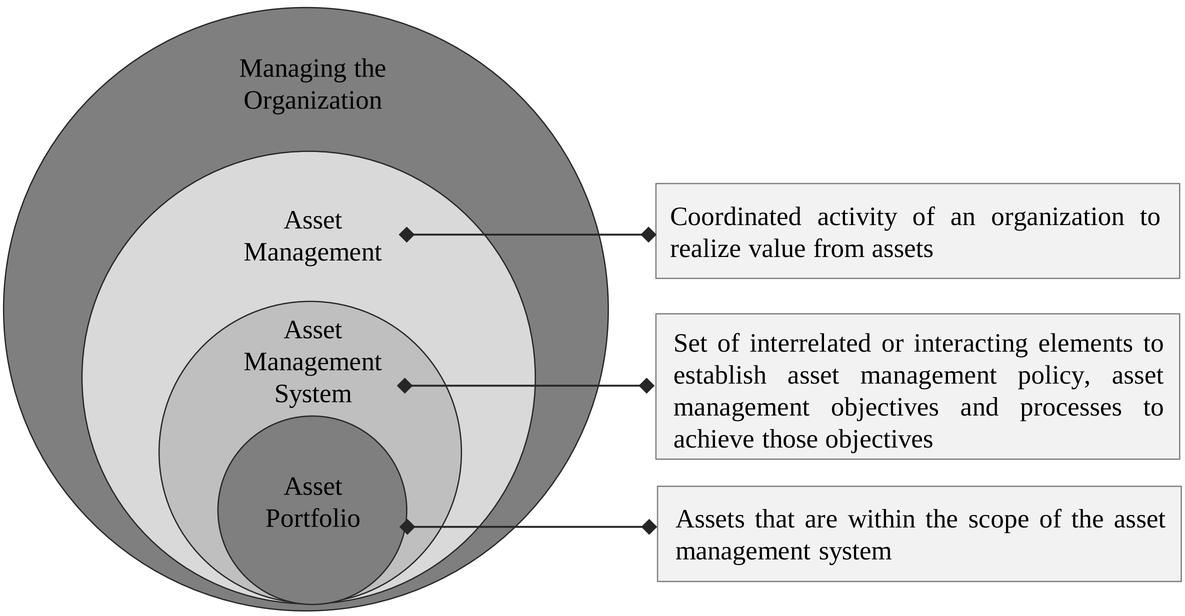


Figure 2: Relationships between key terms in asset management [The British Standards Institution, 2014]

2.3 Overview of asset management approaches

2.3.1 Types and applications of asset management approaches

In the past the term asset management in literature and practice was primarily associated with *financial asset management*. Financial asset management deals with managing and guiding investments for *increased return* purely in financial terms. Infrastructure asset management is also concerned with returns on investment, but it *focuses* on the *whole life of physical assets* and calculates value in terms of the optimum trade-off that can be achieved between social, environmental and economic objectives [Lloyd, C. 2010]. Figure 3 provides an overview of financial asset management and infrastructure asset management regarding their *main focus*, *typical assets*, *system configuration*, *cash flow*, *framework for condition assessment*, *market situation*, *responsibility* and *main stakeholders*.

Financial asset management focuses on maximizing returns by increasing the value of the asset portfolio (cash or cash equivalent). The assets within a portfolio are generally not connected to each other and are therefore *exchangeable*. Despite asset purchases and sales, the immovable property of the portfolio *stays retained* in the long term. Infrastructure asset management, however, aims at proving *functioning physical assets* like roads and bridges within *a certain range of asset condition*. In infrastructure asset management assets as tunnels are generally part of a *functioning system*, like road networks or a route for example. Based on life cycle cost approaches, investment decisions fall between *further expansion of the asset network* and *maintenance of existing assets*. The required budgetary resources are usually covered by *user fees* and *transfer payments* or *taxes*. Whereas in financial asset management the *forecast of the portfolio condition* is essential for an estimation of future returns, the *derivation of financial requirements* for maintaining a certain target condition is crucial for infrastructure asset management. Thresholds arise for both approaches as a result of *risk constraints* and are partially supported by legal restrictions or hedging interests of shareholders and society.

Financial Asset Management	VERSUS	Infrastructure Asset Management
Optimal portfolio management and high return on investment	Focus & Goals	Economically optimal management of functional physical assets
Cash or cash equivalent, equity instruments	Typical Assets	Roads, bridges, buildings, power plants, software solutions
Assets are exchangeable and not connected	System	Assets are part of a functioning system → networks, routes
Asset purchases and sales, fees and taxes	Cash Flow	Expenditures: construction, maintenance Revenues: user fees, transfer of taxes
Investment limits and risk restrictions	Condition Framework	Condition thresholds, legal restrictions
Competitive market	Market Situation	Monopolist situation
Shareholders	Responsibility	Society, shareholders
Customers, asset owner	Stakeholders	Asset users, asset owners, environment

Figure 3: Overview and comparison of financial asset management vs. infrastructure asset management (Own compilation)

2.3.2 Requirements of infrastructure asset management systems

With asset management systems increasingly gaining importance, more and more regulations begin to deal with the standardization of requirements and processes in asset management in order to ensure a minimum quality of methods and infrastructures alike. On a general level, requirements for such approaches depend primarily on the previously specified goals for asset management.

Usable approaches share a number of common characteristics which, in principle, represent a set of requirements for contemporary asset management tools. These characteristics include a *systematic*, *system-oriented* and *multi-disciplinary* method of proceeding. In the context of optimization such approaches attempt to find the best compromise over short- and long-term periods of time *between conflicting objectives*, such as *minimizing costs and risks* on the one hand and *maximizing opportunities and asset performance* on the other hand. Further essential features may include the sustainability of decisions, for example *preserving an optimal asset value* over the *entire life cycle*, with *ongoing system performance*, *environmental* and *long-term consequences* as major aspects. Another characteristic is the probabilistic nature of these approaches as a prerequisite for including risks and reliability in all forecasts. With condition development being associated with the probability of occurrence, informed decisions in this direction are possible. In order to work as a whole, and not as a sum of individual items, the amalgamation of parts requires a comprehensive understanding of the respective system [The Institute of Asset Management, 2014].

Furthermore, holistic approaches should be capable of *converting fundamental aims* of the *organization* into *practical implications* for *choosing*, *acquiring*, *utilizing*, and *maintaining appropriate assets* delivered by identification of an optimal combination of costs, risks, performance and sustainability. For a successful and sustainable management of infrastructure, it is necessary to leave behind popular decision-making processes based on both empirical and qualitative information, and move towards an analytical, *quantitative decision-making process* by implementing appropriate asset management approaches. Such approaches can be considered contemporary if they include *life-cycle* and *risk management methods* and allow the identification of all major decisions and strategies for both individual assets and entire networks. The determination of analytical and quantitative conclusions requires a certain set of data, describing the respective assets. Therefore it is necessary to identify the essential asset information and establish appropriate information repositories [The British Standards Institution, 2014]:

1) **Strategy**: defining corporate levels of service and objectives
2) **Processes**: defining processes with performance objectives, indicators & responsibilities
3) **Asset properties**: function, type, location, condition, age, owner
4) **Asset performance:** service levels, performance, operational requirements
5) **Measures:** types, applicability, dates, impact, costs, duration
6) **Financial management:** service lives, asset replacement value, residual value
7) **Risk management**: failures with probability and consequences
8) **Reporting and feedback**: condition and implementation status
9) **Contract management**: asset related contractual information, third party agreements

Depending on the complexity and expansion of the infrastructure, different data amounts have to be examined, processed and stored. Therefore, handling of the *big-data issue* is becoming increasingly important. For asset management systems this *requires software tools* being able to deal with huge and ever growing amounts of data and transform them into information for decision making.

2.3.3 Basic principles and structure of asset management approaches

Asset management represents a *comprehensive*, structured approach to *long-term optimization* of the *life cycle* of all assets with the main objective of *sustainable value preservation*. Apart from proprietary operator objectives, location requirements and user needs are also taken into account (Figure 4). Asset management approaches are applicable *for all infrastructure operators* (e.g. road operators, railway operators, inland waterway operators and airport operators) and allow an estimation of condition-dependent costs for all relevant infrastructure users or customers (e.g. shipping companies, road & railway operating companies, navigation companies, airlines and port operators).

In addition to *traffic-related stress*, the deterioration of infrastructure condition is influenced by *material parameters* and natural factors, such as *temperature and precipitation*, which substantially *determine structural aging and technical service life*. Since atmospheric conditions usually cannot be controlled, infrastructure engineers have to address weather-driven deterioration processes with proper design solutions and adequate infrastructure dimensioning. As a contributor in the infrastructure condition cycle, infrastructure owners and operators may influence the deterioration process of infrastructure assets by the *implementation of appropriate measures* and thus improve asset condition up to its initial quality level (illustrated in Figure 4). The service life of infrastructure is substantially determined by the means of *sustainable planning and construction* of assets, as well as the effectiveness and efficiency of measures implemented during the life cycle. The prevailing infrastructure condition shows a relevant impact on infrastructure users. In a worst case scenario, the infrastructure condition is decisive, whether or not the infrastructure is considered as an optional transport route. Furthermore, insufficient infrastructure condition may result in negative follow-up costs for infrastructure users due to increased *accident rates*, *congestion*, and *detours* or lead to *load restrictions* on transport routes with increased transport costs in the respective corridor.

Figure 5, Figure 6 and Figure 7 provide a compilation of main interdependencies between *infrastructure condition*, *maintenance activities of infrastructure operators* and the resulting *impacts on infrastructure users*. In addition to infrastructure operation costs, the presented deterministic life cycle considers different rehabilitation strategies and their impact on deterioration development. The implications of these rehabilitation strategies for operators are thereby illustrated through a cumulative cost development over the life cycle.

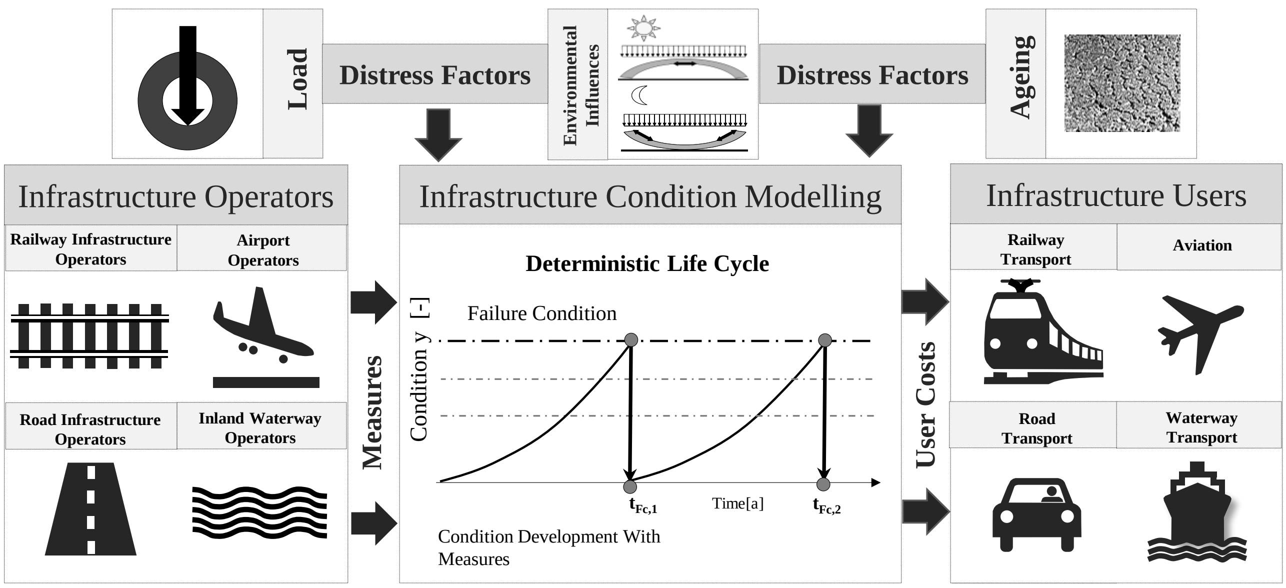
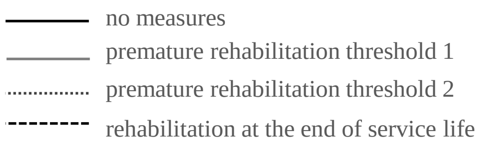
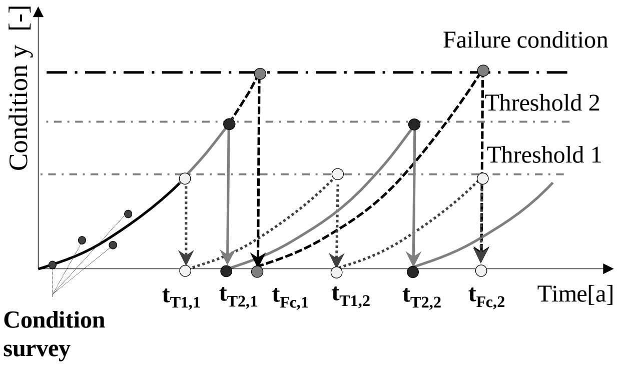


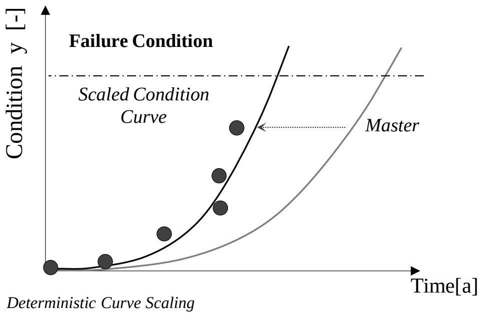
Figure 4: Infrastructure condition (illustrated by a deterministic life cycle with replacement strategy) is affected by distress factors such as traffic load, temperature and ageing processes, which lead to the deterioration of asset condition. Infrastructure operators strive for condition improvement through implementation of appropriate measures, resulting in improved infrastructure conditions for users. (Own compilation)

Deterministic life cycle for different replacement strategies





Condition survey & curve fitting



Data collection and curve fitting:

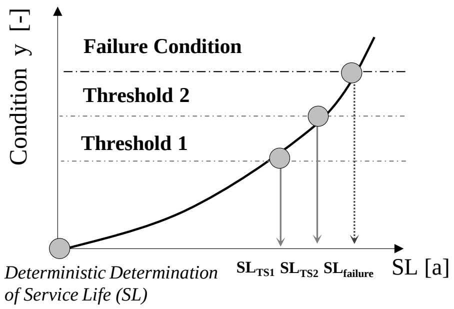
<u>Survey methods:</u>

- Visually sensitive condition survey
- Visually manual condition survey
- Automated condition survey

<u>Curve fitting methods:</u>

- Curve shifting (horizontal / vertical)
- Curve scaling
- Det. regression (linear,…)
- Prob. regression (bivariate, multiple,…)

Infrastructure condition assessment



Condition assessment:

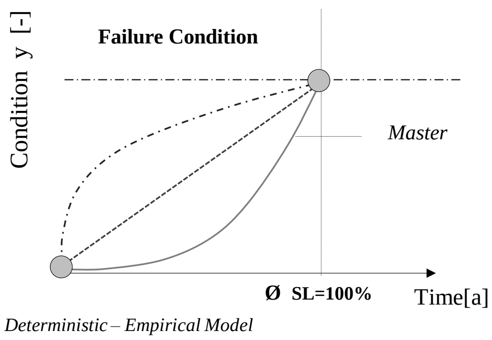
<u>Typical assessment procedure:</u>

- Condition transformation (rating functions, threshold value,..)
- Condition standardization (weighting, summation,..)

<u>Determination of service life</u>

- Deterministic models (average service life)
- Probabilistic models (failure distribution)

Infrastructure condition prediction



Condition prediction models:

<u>Empirical / mechanistic models:</u>

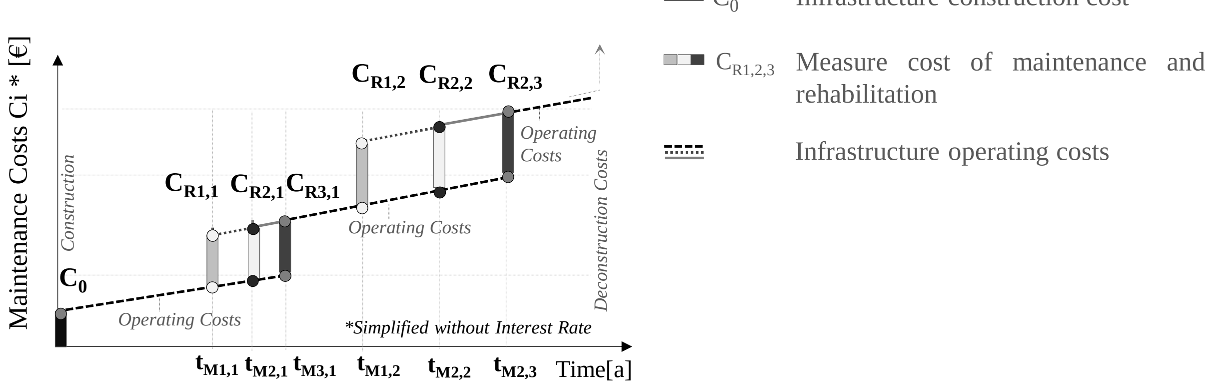
- Empirical (E) – models
- Mechanistic (M) - models
- (E-M) - models
- (M-E) - models

<u>Deterministic / probabilistic models:</u>

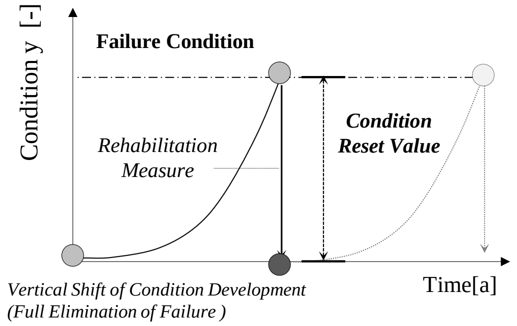
- Deterministic models
- Probabilistic models

Figure 5: Basics of deterministic infrastructure condition modelling including condition survey, condition assessment and condition prediction for different rehabilitation strategies; own compilation based on [Hoffmann, M. 2009]

Simplified cost cycle from operators‘ perspective



Occurrence of measure costs



Condition development with measures:

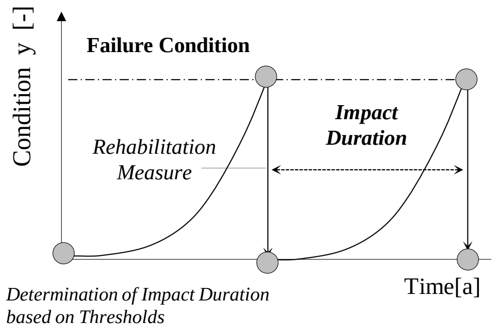
Reset value element condition:

- Matrix with impact of different measures (reset values) on different failure types

Condition development after measure:

- Vertical shift and condition prediction with initial failure performance curve
- Parallel vertical shift of initial failure performance curve
- Accelerated failure progress after measure

Duration of measure impact



Duration of measure impact:

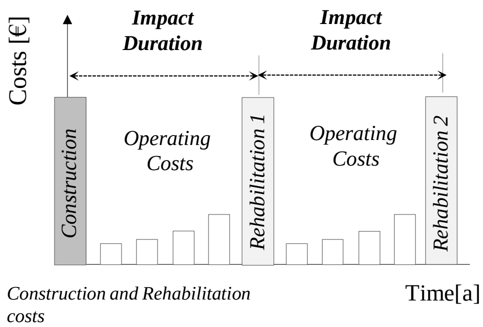
Methods of determining impact duration:

- Expert opinion (missing basic data)
- Lab testing / field testing
- Based on average intervention frequency
- Based on thresholds
- Based on condition before measure

Impact duration is effected by :

- Load, temperature
- Condition before measure

Occurrence of measure costs



Measure costs during life cycle:

Operator Costs during life cycle:

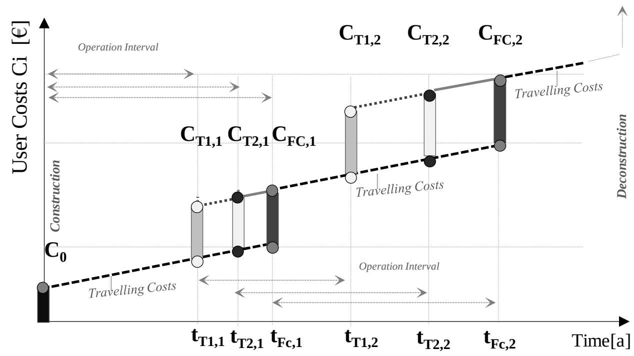
- Planning / construction
- Operation
- Maintenance / rehabilitation
- Deconstruction

Methods in cost estimation:

- Top-down
- Bottom-up

Figure 6: Basics of life cycle cost modelling from infrastructure operators’ perspective including condition reset value of measures, impact duration of measures and typical measure costs during asset life cycle; own compilation based on [Hoffmann, M. 2009]

Simplified cost cycle from users‘ perspective

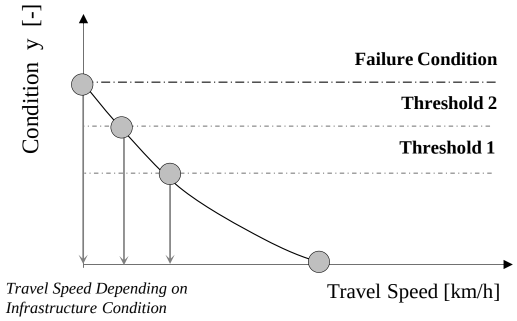


C_0 Time loss, detour costs due to construction

$C_{T1,2,FC}$ Congestion costs for users due to rehabilitation measures (time loss,..)

Average user costs for travelling

Impact of infrastructure condition on users I



Condition impact on users:

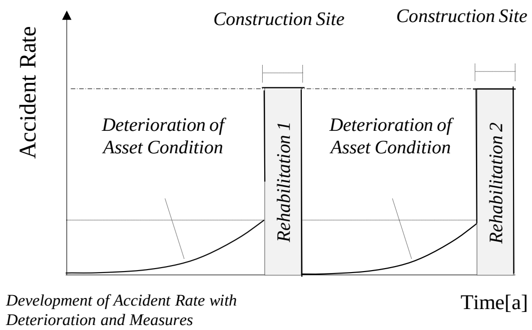
Impact on travel speed:

- Travel speed is directly related to infrastructure condition and decreases with increasing deterioration
- The indicator variable (characteristic failure mode) depends on the infrastructure

Impact on load:

- Possible load restrictions (fleet constellation and utilization)

Impact of infrastructure condition on users II



Asset condition and accident rate:

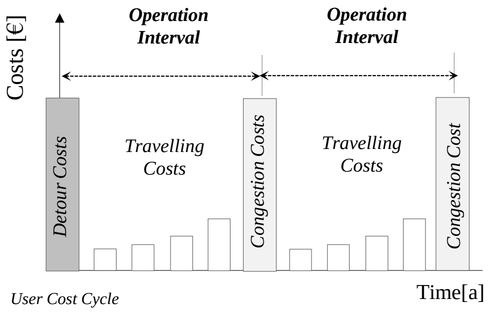
Condition induced accidents:

- Increasing accident rate due to progressing deterioration

Construction related accidents:

- Construction sites due to measure implementation increase the accident rate rapidly
- The duration of construction or maintenance works extends the period with increased accident rate

Cost Cycle Infrastructure Users



User costs during life cycle:

Congestion and detour costs :

- Detour costs, time loss during construction
- Congestion costs due to maintenance
- Detour due to non available route due to deconstruction

Travelling costs

- Variable (fuel, toll, time, …)
- Fix (amortization vehicle, repair costs, fees,…)

Figure 7: Basics of life cycle cost modelling from infrastructure users’ perspective including impacts of infrastructure condition, and typical user costs during asset life cycle; own compilation based on [Hoffmann, M. 2009]

Figure 5 summarizes the most important components of condition modeling, starting with the selection of the *prediction model* based on surveyed condition information. The main approaches in condition survey contain *visual and automated methods*, which provide the basis data for condition modelling. With the *master curve* as average characteristic of condition development at hand, an adaption to collected condition data for individual assets and elements is carried out, e.g. *using shifting, scaling or regression methods*. Figure 5 addresses the steps in the typical process of condition assessment and the impact of *condition thresholds*. As condition thresholds determine the service life of infrastructures and impact duration of measures, a careful and comprehensive evaluation of thresholds as well as possible failure impacts on users, operators and the environment is in order.

Furthermore, the resulting maintenance and rehabilitation costs for infrastructure operators are presented following the rehabilitation cycle, illustrated in the condition modelling section. Apart from *planning and construction costs*, infrastructure owners must provide a budget for *operational* and *structural maintenance activities*, *rehabilitation measures* and *asset deconstruction*. Whereas maintenance and rehabilitation measures arise with their cost amount at the time of measure implementation as a vertical shift of the cost curve, operating costs have to be considered continuously during service life in cost calculations. The cost cycle for infrastructure operators includes operating cost illustrated as a linear increase in the cost curve between structural measures. In order to optimize infrastructure cost for the entire life cycle, it is necessary to determine both, *measure impact* on *condition development* (*condition reset value*[7]) and the *duration of measure impact* as well.

For infrastructure users, the cost cycle includes average user costs for *travelling (fuel, toll, and amortization costs)* as well as *congestion* and *detour costs* (Figure 7). Congestion and detour costs occur especially due to *construction sites* as a result of measure implementation. The average condition development of infrastructure assets affects user safety as an increased condition deterioration results in an increase of the average accident rate. In addition to a condition-related decrease of transport speed, a poor infrastructure condition may lead to load restrictions, which in turn also affects user costs [Hoffmann, M. 2009].

As discussed previously, there are a *number of stakeholders*, which are *affected* by asset management processes; in practice stakeholders as environmental organizations, infrastructure users, and infrastructure owners have a number *of conflicting requirements and goals*, which partly exclude each other. Inherently, the provided infrastructure must offer a certain quality and capacity at a high level of user safety combined with a high reliability and availability to attract users and thus compete on the transport market.

Accessible information on infrastructure availability is an essential prerequisite for a functioning and competitive infrastructure. An increase in pressure towards reduction of negative environmental impacts due to stringent climate targets and environmental organizations, which have gained importance in recent years, naturally raises costs for infrastructure construction and maintenance, or results in costs for compensatory measures.

Thus, in practice budgetary, shortfalls arise for many infrastructure operators, since possible fees are limited and they have to meet quality claims of customers as well as the demand for more environmentally friendly infrastructures. As a result, *high technical and environmental standards* may lead to conflicts with *short-term economic goals* of infrastructure operators and owners that have to be resolved as well.

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7 A condition reset value of zero means that the measure has no impact on the respective failure type. Typically reset values for single failure types are defined for each measure as a part of the measure catalog. (also see HOFFMANN, M. 2009).

Generally, the basic structures of available infrastructure asset management approaches are quite similar to each other, regardless of the type of considered assets, and follows uniform standards and principles. Moreover, infrastructure asset management can be described as a complex, dynamic and cyclical process, which includes a number of core modules and standard procedures (Figure 8). In order to *preserve the value of infrastructure assets* at the *most sustainable level possible*, life cycle *cost approaches are used for measure selection* and optimization accounting for restrained budgets. Optimizing the overall infrastructure asset management process is only possible with ongoing development and improvement of individual steps in relation to overall objectives of the infrastructure operator and stakeholder engagements. With feedback loops, self-learning systems support the ongoing optimization process using current condition and cost data.

Figure 8 provides an overview of all of the important modules and basic processes of a comprehensive asset management approach. Each systematic asset management approach starts with the *defining of goals* and the establishment of an asset management strategy in accordance with the *company's objectives* and the general asset management policy (see chapter 2.3.4).

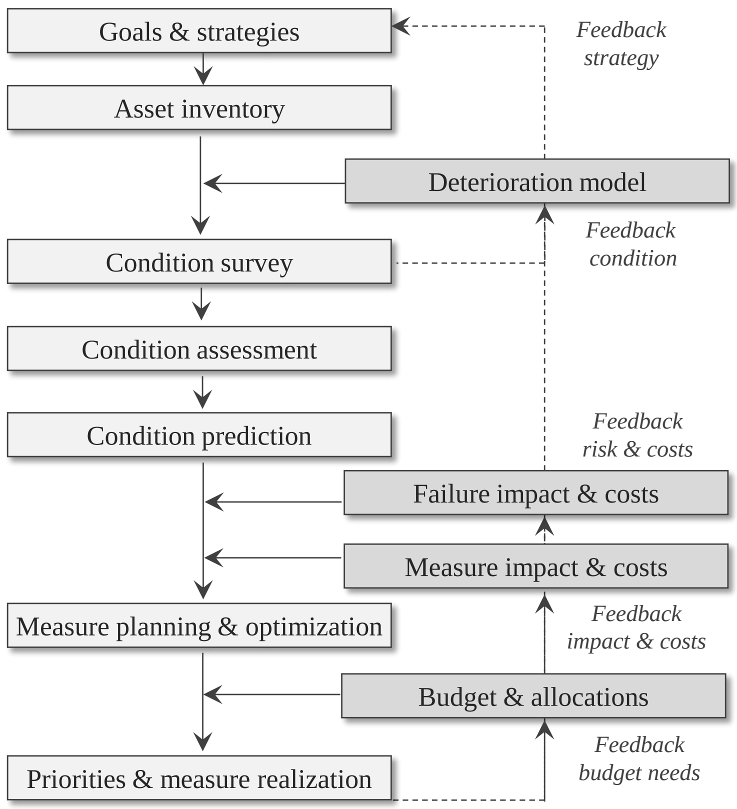

Figure 8: Overview modules and standard procedures in infrastructure asset management; own compilation based on [Hoffmann, M et al. 2012b]

The next step is to identify, detect and store all essential infrastructure elements starting at the network level down to individual assets and asset elements, including their logical horizontal and vertical configuration in an *inventory database* (see chapter 2.3.4.1). This core database includes all essential parameters which are subsequently needed as input parameters for any kind of life cycle approach.

At the asset element level, a number of *deterioration functions* describing different failure types are defined, which in turn determine the condition of individual assets in an aggregate form so that an overall performance at the network level can be characterized by logical linking of individual assets (see chapter 2.3.6). These deterioration functions provide the basis for condition prediction and condition assessment. Within the scope of health monitoring, existing damages are detected periodically at element levels including *damage extent* and *severity* (see chapter 2.3.8). With an increase in the number of surveys, the accuracy of condition prediction can be enhanced.

For *condition prediction*, both deterministic and probabilistic methods are available (see chapter 2.3.6). Although deterministic methods are easier to handle, several inaccuracies are limiting their applicability. By *establishing condition thresholds, and functional requirements* the calculated service life of elements is determined (see chapter 2.3.9).

Prior to measure planning, it is necessary to define a *standardized catalog* for all *applicable measures* and their impact *on individual failure types*. These measures improve the condition of asset elements for different deterioration functions to a certain amount (condition reset value) and for a certain time period (impact duration).

Furthermore, costs incur for each measure, illustrated by cost functions depending on measure extent accounting for economy of scale. Based on these cost functions, reasonable cost estimates can be established for planning and preparation of any kind of measures. Hence, the optimization of measure planning uses life cycle cost approaches with measure costs, impact duration and interest rate as central input parameters (see chapter 2.3.14). Due to tight budgets, a full implementation of a construction program resulting from an optimization process may not be feasible for many infrastructure operators and therefore requires a ranking of measures within available budget (see chapter 2.3.15). The ongoing integration of implemented measures ensures up-to-date data bases and allows a continuous verification of achieved objectives.

2.3.4 Goals in infrastructure asset management

From the strategic perspective, the *principles of asset management* must be in alignment with the *purpose* and *resources* of an infrastructure owner or operator. Under market conditions, the *needs of customers* have to be met in order to maximize returns under the constraints of the respective regulatory environment and legislation. In general, the goals of infrastructure asset management are *long-term oriented*, meaning that sustaining assets are often in conflict with *short-term optimization* of *cash flows* or shareholder values. Apart from safety regulations and constraints, turning over actual revenues to shareholders might be favorable for these shareholders and a possible bonus for managers, but can lead to insufficient funds for necessary future investments. Thus, there are a number of conflicts between different stakeholders in the management of infrastructure assets that may lead to different strategies and results.

Figure 9: Overview of typical goals in engineering asset management; adapted from [Amadi-Echendu, J. et al. 2010]

According to AMADI-ECHENDU[8], infrastructure asset management needs to address the development of asset value, safety and resilience in addition to the legitimate interest for reasonable profits of shareholders. Furthermore, the interests of other stakeholders have to be considered as well, especially if an asset management is to be implemented for public infrastructure assets. Figure 9 provides an overview of typical goals in infrastructure asset management, which generally aim at market competition and steady revenues [Amadi-Echendu, J. et al. 2010 and Hoffmann, M. et al. 2012b].

[8] Definitions, concepts and scope of engineering asset management, 2010

1) **Capacity matching**

Successful infrastructure operators and owners must provide a competitive infrastructure with a high capacity and reliability. In order to enable an economical operation of assets, infrastructure offers must avoid over- or under-capacities in terms of market requirements. The provided capacity should be scalable based on analysis and prognosis of demand and requested services. Providing infrastructure with a capacity far exceeding actual demand is inefficient, because possible revenues from better investments cannot be realized. On the other hand, low capacities and bottlenecks may contribute to low satisfaction of customers, losses of market shares and the economy in general. A holistic asset management approach would allow for optimization of existing capacity relative to both actual and future demand.

Typical assessment criteria include: *Level of Service, utilization, and waiting times.*

2) **Meeting customer needs**

In order to remain competitive on the market, the infrastructure must provide a higher value to customers compared to other alternatives. The value for customers can be enhanced by meeting important infrastructure requirements, such as high quality, availability, reliability, safety, accessibility, environmental sustainability, compliance and timeliness. With infrastructure operation as core business, a high availability, reliability and safety of infrastructure assets for customers have proved to be essential. Asset management helps to balance these aspects (e.g. in order to avoid losing customers and market shares due to insufficient investments in safety and reliability).

Typical assessment criteria include: *Availability, reliability, safety, accessibility, and timeliness.*

3) **Preserve asset value & cost efficiency**

Infrastructure owners and operators have to decide carefully how to spend their scarce resources. Therefore, implemented measures should achieve a high level of efficiency and effectiveness. Retaining the value of engineering assets is associated with investments in improving asset condition and performance as well as an extension of service life. Common methods use costs and impact durations as input parameters for a comparison of investment options. In order to improve the cost efficiency in asset management, over an entire life cycle it is necessary to integrate life cycle cost models and their results in decision making.

Typical assessment criteria include: *Investment needs, actual budget, annuity, present value, condition performance and remaining service life.*

4) **Market leadership**

In order to be competitive in the market, infrastructure owners and operators need to meet customer requirements. On an individual basis, the customers decide which alternative best suits their needs. Therefore, an increase in market shares as a sum of individual decisions can be seen as an indicator for customer satisfaction, or at least a better performance, when compared to other alternatives. Whether or not such a market leadership can be translated into a steady flow of revenues depends on the efficiency of investments and the ability to raise fees and taxes or gain access to subsidies. Furthermore, setting standards and being innovative are important factors to remain competitive in the future as well.

Typical assessment criteria include: *Growth forecasts, market shares, revenues, costs and profit.*

2.3.4.1 Levels of decision making and information needs

In order to realize all objectives within the asset management framework and manage infrastructure assets successfully, decisions on different hierarchical levels, such as *policy level*, *network level* and *project level* are required. With increasing hierarchical level, the level of necessary information detail is decreasing. Thus, decisions on the asset management policy, including the *vision for the infrastructure assets*, *core principles* (e.g. commitment to minimum life cycle costs and sustainability) and *main implications* for customers and stakeholder, are taken at the strategic level, based on current reporting of subordinated levels. At the network-level the asset management strategy is defined, including *target infrastructure performance* (capacity, condition, availability), according to the policy vision as well as a strategy for main tasks and their practical implementation. The derivation of assets, or locations with necessary measures, is a result of an initial prioritization process with the focus on effective and efficient measures aiming at achieving the highest possible network-wide infrastructure quality. Maintenance strategies at the network level are extrapolated based on data from a comprehensive AM-database. By detecting traffic load, bottleneck evaluations are possible as well as categorizations of traffic importance. Thus, future needs for infrastructure development may be derived on the one hand and appropriate safety systems may be established on the second hand.

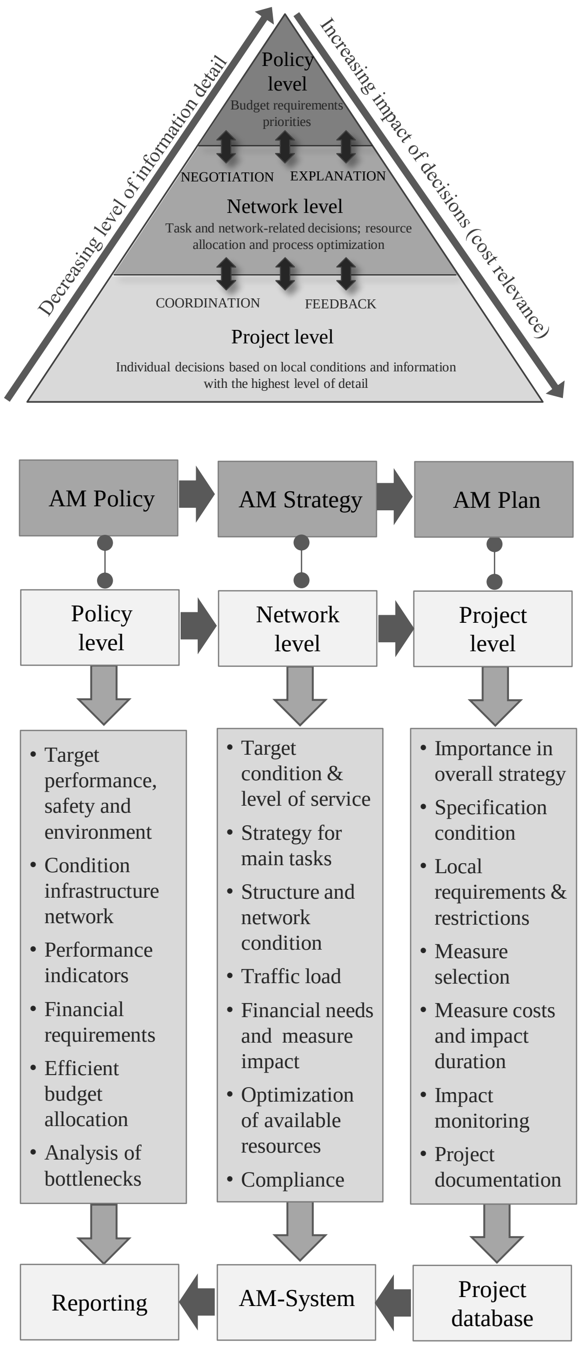

Figure 10: Levels of information and communication in infrastructure asset management; own compilation based on [Hoffmann, M 2009]

However, with decreasing hierarchical level, the impact of decisions on tangible results is becoming more immediate. In order to achieve previously specified local target conditions at project level, the selection of appropriate measures is supported by information of local requirements and restrictions. The selection of the most efficient measure for specific assets with measure costs and measure impact on condition is based an LCC-based optimization approaches. Furthermore, the information describing the effectiveness of measures is collected at project level as a part of a continuous monitoring process. The project documentation forms the basis for *subsequent controlling processes* and ensures the *dissemination of results and key performance indicators* for future planning.

2.3.5 Asset inventory

A *database-supported inventory* of all essential assets and asset elements signifies an important step for setting up any asset management solution. This *core database* thus not only represent the basic structure of each asset management, but also defines the logical vertical and horizontal linkage of all major components of the respective system (Figure 11). At the top level, a horizontal connection of individual assets forming a network is possible either by serial or parallel configuration. Each individual asset in turn consists of a number of elements. Both individual elements and individual assets may prove to be critical for the function of the entire system, either because they are not redundant, have a very short service life or are associated with high failure costs.

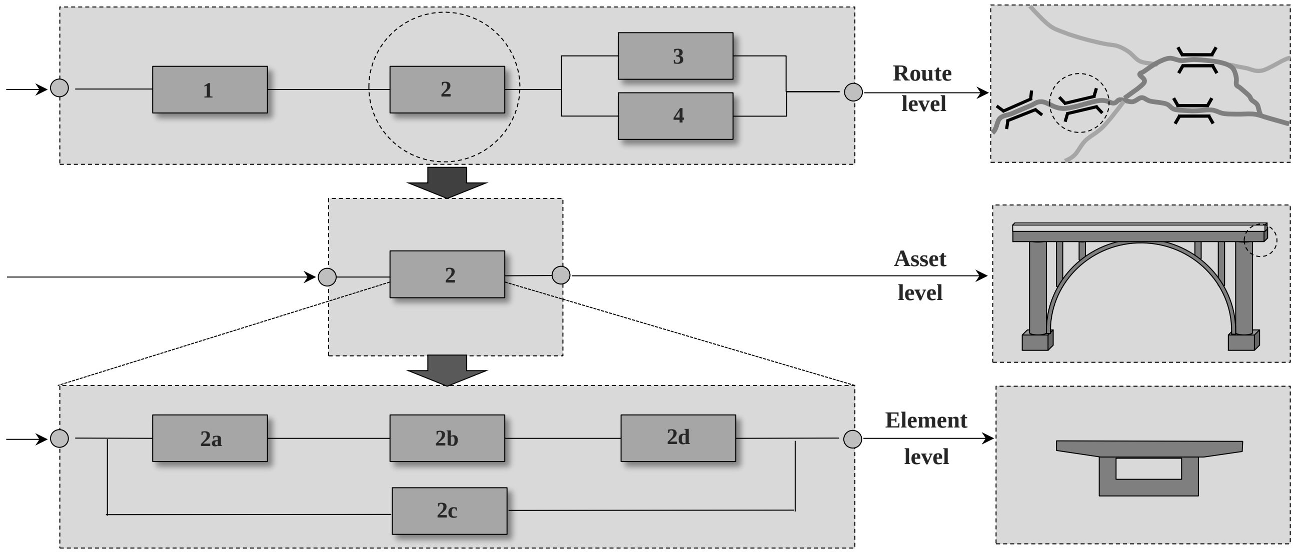

Figure 11: Hierarchical structure of asset elements, assets and routes; own illustration based on [Hoffmann, M. 2009]

The positioning of individual assets within a network (*localization*) is essential for manageability, especially for systems with large amounts of expansion and a complex structure. Currently, the localization of assets is usually achieved using *GPS*. In many areas of infrastructure asset management, common database solutions offer interfaces for geographic information systems (*GIS*[9]). The localization of infrastructure assets is followed by the allocation of assets to routes (*routing*) corresponding to the horizontal linkage of assets and has a strong impact on the reliability of the system. Furthermore, the configuration of assets determines further performance indicators such as availability, maintainability and safety. All elements surveyed in the context of asset inventory may then be visualized on maps in order to provide an overview of the relevant infrastructure.

Particular attention should be given to the preparation of the core database of assets, as the assignment of all relevant parameters of life cycle costs and risk approaches to asset elements is already defined at this stage. Thus, data storage should include specifications of *asset age*, *average service life* and *construction type* as well as *failure probability* and *responsibility*. Comprehensive data management is crucial for the resulting quality and success of asset management tasks in the long term. This refers to *constant updating* of all essential basic data describing *where, how and why something* is happening. In doing so, it is possible to ensure that investment decisions are based on current data and that database queries can provide a conclusive illustration of asset performance.

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[9] *Geographic information systems are designed to capture, store, manipulate, analyse, manage and present all types of spatial or geographical data.*

2.3.6 Introduction to condition modelling

Unfortunately, infrastructure assets do not remain in their initial condition after their construction, but are subject to an ongoing *deterioration process* which is determined by factors such as *environmental impacts*, *loading* and *structural aging*. Deterioration is a *time-dependent* process and describes the mechanism by which assets deteriorate and pass through different stages until failure. The increasing extent and severity of damage could either proceed gradually over time or occur in discrete steps [Abra, E. 2012].

Figure 12 provides an overview of the systematics of condition modelling for single elements with several failure types, infrastructure assets and entire infrastructure routes. Deterioration is a process occurring at *element level* with a certain number of *deterioration functions* per element (e.g. pavement surface, Figure 12, element number 2d) accounting for *different failure types* such as rutting ($2d_1$), delamination ($2d_2$) or longitudinal cracking ($2d_3$).

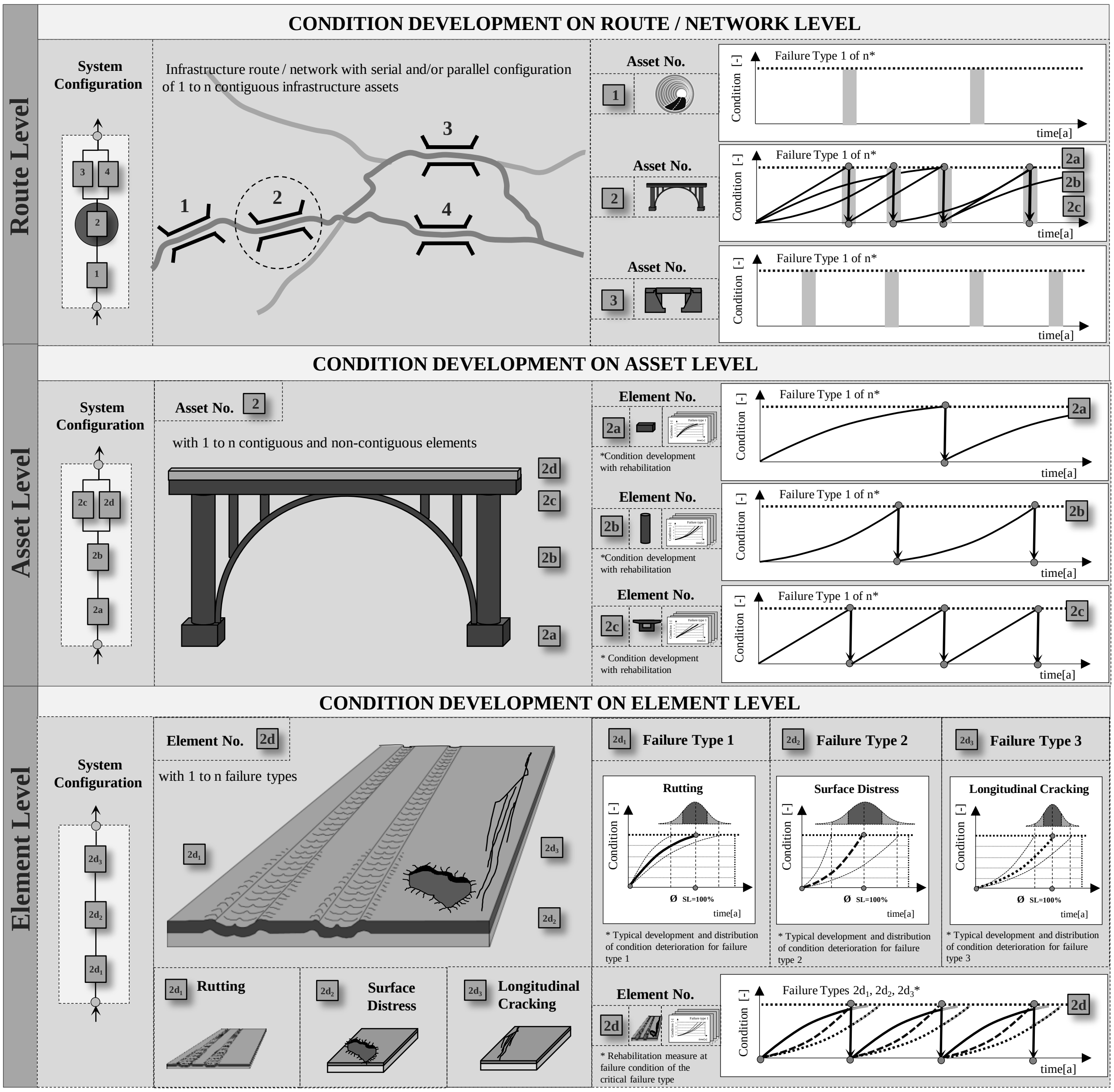

Figure 12: Condition deterioration relates to asset elements, where failure extent increases over time, with 1 to n deterioration functions for each element accounting for different failure types; own compilation based on [Hoffmann, M. 2015]

On element level, master condition functions of typical failure types are illustrated together with standard deviation and confidence intervals. When an element is considered, which comprises a certain number of deterioration functions ($2d_1$, $2d_2$, $2d_3$), usually the failure type with the most *progressive deterioration curve* and/or service life (critical failure e.g. $2d_2$), is decisive for the application of any appropriate measure (deterministic approach). As a result of measure implementation, the condition development of other failure types may be improved as well depending on the type of applied measures in order to avoid failures. The compilation of all deterioration modes of all elements to a *failure catalogue* may be described as the centerpiece of any systematic asset management approach.

The same principle also applies to the condition development on the asset level. An aggregated condition development on *asset level* is, in turn, a result of a *summarization of condition curves* of individual elements (2a, 2b, 2c, 2d), whereas the *element* with the *most critical deterioration* development implies the application of a measure (deterministic). However, from a stochastic point of view, time until failure may vary e.g. leading to an in average shorter service life for serial systems compared to the most critical element or failure type (stochastic) [Hoffmann, M. 2009]. The harmonization of both respective appropriate measures for single elements as well as intervention times, represents a fundamental part of the optimization process in asset management (also see chapter 2.3.16).

Infrastructure routes and networks contain a number of connected infrastructure assets (Figure 12, assets no.1, 2, 3). At the *network level* an overall performance, in terms of reliability, availability and safety, may be determined. The system configuration of single assets (parallel or series) plays a crucial role for the evaluation of risk and reliability of a network. Due to the large scope and complexity of reliability mathematics, the aspects of system reliability and failure risk may be considered only peripherally within this thesis. The following section of this thesis will provide an overview of the most common mathematical functions and methods that are used in condition modelling.

The deterioration process is usually described by mathematical functions and different types of prediction models. Generally, there are different concepts available to depict the issue of deterioration. These concepts contain *general condition development*, *total performance development* and *failure risk development*. Therefore, it is important to have a clear distinction between major concepts, the concept of condition development (upwardly curved) and the idea of performance modeling (downwardly curved). Whereas the *condition of elements* is described by parameters that are actually *measured*, the description of *element performance*, however, always results *from previous condition assessmen*t. The deterioration of failure severity (e.g. width, single cracking in mm) and failure extent (e.g. area of cracking in m²) is subject to a specific unit and illustrated by *upwardly extending conditions function*s (Figure 12) with severity and extent increasing over time if no maintenance or rehabilitation measures are implemented. These averaged condition functions act as *master curves*[10] and are calibrated for single elements to match data from observed conditions.

Apart from condition functions, which are based on quantifiable measured condition values, another common concept *consolidates* individual failure types by *weighting* and *grading* (1=very good; 5= failed) in order to determine a total condition performance. The rating of the actual and future condition is based on a rating background translating actual values into grades and provide an overview where and which failure extent might be critical or not. Such *rating backgrounds* have a significant impact on condition development

[10] Master curves can be derived from laboratory tests or long-term observations. They describe a set of theoretical curves, calculated for known models (e.g.by lab testing) and may be compared to field curves indicating whether they are reasonable or not.

and the resulting derivation of appropriate measures. According to HOFFMANN[11], the common *aggregation* of *condition functions* to an condition index by *weighting* always results in a loss of information regarding *the underlying deterioration causes* and their progression over time. A well-known example for such a concept in infrastructure engineering is the *pavement condition index* (PCI [12]). This numerical index ranges between 0 and 100 (where 100 represent the best possible performance) and indicates the general condition of the pavement. With increasing pavement age, this index decreases, resulting in a downward curve for general performance (= *degradation curve*). Furthermore, deterioration may be described using failure probability of elements, which is increasing with a rise in system age.

The rate of deterioration is graphically described by a master curve, where the slope of the curve indicates the durability or robustness of an asset. To ascertain which *mathematical form* is most appropriate for a given set of condition data, the raw data must be plotted so that the resulting scatter diagram can be compared with standard curves (see also Figure 13) ranging from *linear*, *polynomial*, *exponential*, and *logarithmic* to *power functions* [Labi, S. 2014]. These functions differ regarding the number of freely selectable parameters and their flexibility concerning the adaption to measured condition data. The adaption of these mathematical master functions to actual condition data is usually based on methods of standard regression analysis (see chapter 2.3.8).In infrastructure asset management the shape of the condition curve is essentially determined by the *underlying failure mechanism.* A *linear condition development* occurs if, for example, failures increase continuously over time. Fatigue failures, however, are characterized by a *progressive condition development*, meaning that failures are progressing at an accelerated rate with the first signs of damage becoming visible. *Digressive condition development* appears after strong initial damages (such as settlement) and usually progresses very slowly.

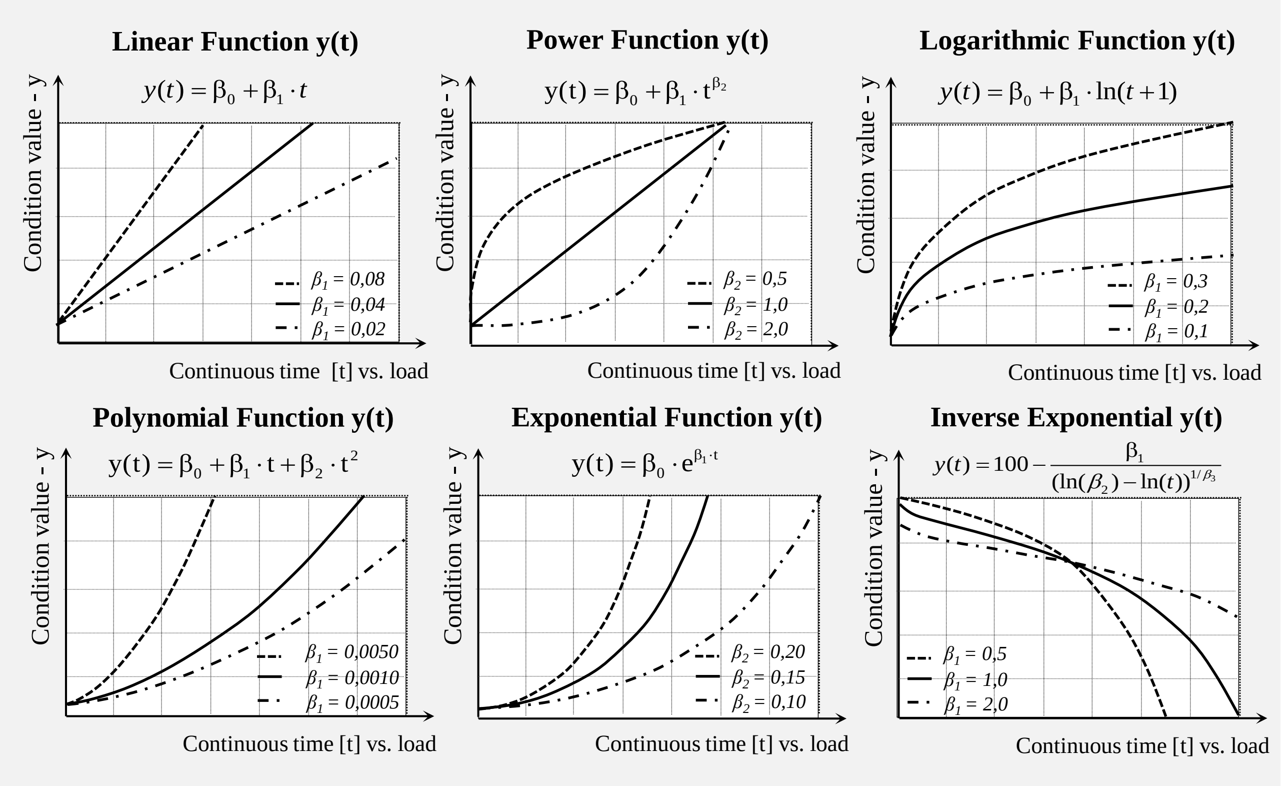

Figure 13: Common mathematical master functions used in condition modelling [Labi S. 2014] and [Donev, V. 2014]

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[11] *Comparison of pavement condition assessment and prediction models on road section an network level; PIARC 2015*

[12] *http://hawaiiasphalt.org/wp/wp-content/uploads/PCI-101.pdf; [Last access: 15.03.2015]*

2.3.7 Condition prediction models

Predictions or forecasts are statements about future events and are usually *experience- or knowledge-based*. In infrastructure asset management, prediction models might be oriented either on short-, medium- or long-term perspectives, depending on the average service life of considered structures. As a function of *available basic data,* and whether *uncertainties* are included or not, a number of models may be applied methodologically for the description of the deterioration process. These mathematical models can be classified regarding certain characteristics. Prediction models, which are used for infrastructure assets are classified according to Figure 14:

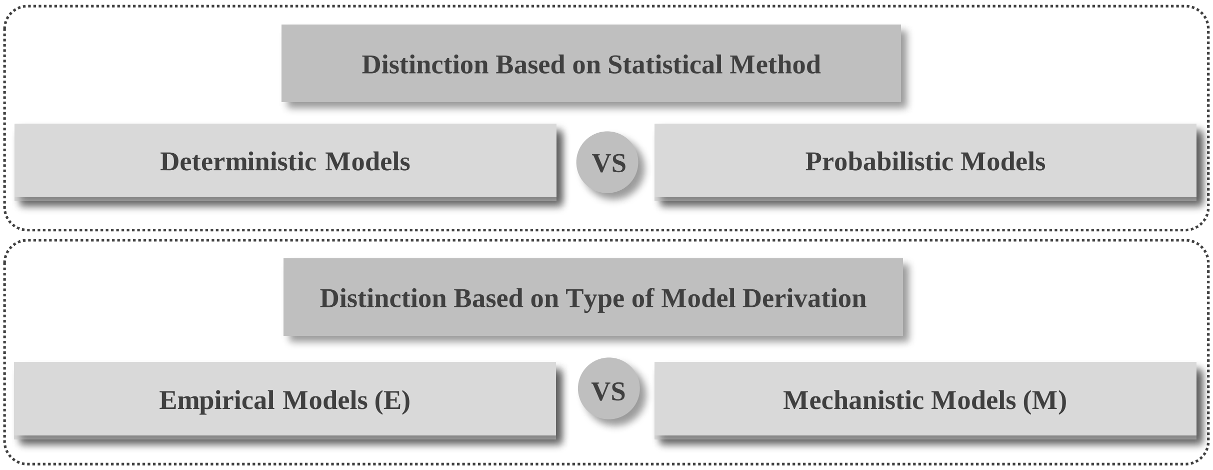

Figure 14: Classification of major prediction models for infrastructure assets.

2.3.7.1 Empirical models (E-Models)

Empirical models are developed by relating *condition scores* to *explanatory variables,* such as *age* or *loading conditions*, usually through a *regression process*. These models are usually based on experimental data (e.g. field experiments). Empirical models have the advantage that the adopted condition development can be very flexibly; however, the cause of a failure type remains a *"black-box"*[13]. This implies that the underlying deterioration process is either *unknown* or difficult to describe. In common models, the condition development is related to one single impact factor (usually time). Moreover, the mathematical function (see Figure 13) which fits best to the data (with the highest coefficient of determination) is selected for condition prediction [Hoffmann, M. 2009].

If more than one explanatory variable is influencing the condition variable, *multiple regression* is the most common method used to depict fundamental connections. By minimizing ordinary least squares of deviations, the model parameters are estimated (e.g. linear multiple regression):

$$\hat{y} = b_0 \cdot x_0 + b_1 \cdot x_1 + \ldots + b_k \cdot x_k \tag{1}$$

where $b_0, b_1, \ldots, b_k$ are estimating the regression coefficients, $\hat{y}$ is the predicted value of the dependent variable, and $x_0, x_1, \ldots, x_k$ are values of *independent variables*.

[13] *No prior model is available; black-box systems can be viewed in terms of its inputs and outputs, without any knowledge of its internal workings. The black-box theory is based on the explanatory principle, the hypothesis of a causal relation between input and output; also see: https://en.wikipedia.org/wiki/Black_box; (Last access: 15.03.2015)*

In the case of infrastructure deterioration, $\hat{y}$ describes the condition of an element, and $x_0, x_1, \dots, x_k$ are parameters affecting the condition of the element, such as element age. The method of *least squares* is the most common form used to derive the coefficients $b_0, b_1, \dots, b_k$ [Abra, E. 2012]. Empirical models are universally applicable, and may be implemented within a short time frame with minimal effort and low costs. Since empirical models are derived for specific framework conditions, their *validity* is *restricted*.

2.3.7.2 Mechanistic models (M-Models)

Mechanistic condition prediction models link *material behavior*, derived from *laboratory tests*, with the *response behavior* of *numerical modeling*, in order to describe the process of deterioration. Since the underlying physical process is clearly described, mechanistic models are often referred to as *"white-box"*-models[14]. Mechanistic models incorporate physical processes and therefore allow more qualified statements about future developments. However due to deterioration being usually caused by an interaction between different factors, mechanistic models fall short of complex failure modes and usually show deviations to actual behavior and service lives, as implemented failure mechanisms must be simplified. Furthermore, the derivation of valid mechanistic condition prediction models and their calibration to match actual performance and service lives is described as a costly and lengthy process. However, for the *basic type* and shape of the *master performance curve* (see Figure 13), mechanistic models can provide *valuable indications* [Hoffmann, M. 2009].

2.3.7.3 Combined models M-E and E-M

Mechanistic-empirical models as well as empirical-mechanistic models are increasingly gaining importance in infrastructure engineering. Such models are based on both mechanistic and empirical principles are known in "system theory" as *"grey-box"*-models[15]. Thus, for M-E-models, material parameters (from laboratory tests) and traffic loads may serve as input parameters for finite element models and thereby allow an analytical determination of the deterioration process. As a result, the calculated service life can be used for the calibration of empirical condition functions. For E-M-models, condition functions and explanatory variables are selected based on system-theoretical considerations, while their model parameters are derived from empirical data by statistical methods. An advantage of this prediction method is that less asset condition data is needed [Donev V. 2014]. According to state-of-the-art literature in condition modelling the combination of empirical and mechanistic principles provides the most reliable condition predictions.

2.3.7.4 Deterministic models

Deterministic prediction models for infrastructures are based on the assumption that all factors affecting the deterioration process are *well known*, *quantifiable* and *measurable*. In many cases, deterministic models have replaced the long-established expert opinion. Deterministic models are typically illustrated by performance functions and output a single condition value for a given input parameter. Although deterministic models with a single prediction value at time are easier to handle, they are almost always wrong because it is in fact nearly impossible that exactly this single predicted condition value occurs in practice.

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[14] *White-box models are based on a first principle (e.g. a model for a physical process from the Newton equations); in many cases such models are impossible to obtain in reasonable time due to their complex nature; also see: https://en.wikipedia.org/?title=System_identification'; (Last access: 15.03.2015)*

[15] *Although the peculiarities of the process inside the system are not entirely known, a certain model, based on both, insight into the system and experimental data, is constructed; see: https://en.wikipedia.org/?title=System_identification; (Last access: 15.03.2015)*

Contemporary deterministic approaches for condition prediction in infrastructure modelling often use "*power functions*" (see Figure 13 and Figure 15) to describe the deterioration process. The average time to reach a specific condition is calculated based on individual master curves for each failure type and has to be fitted to match age and actual condition. In the presented prediction model β_2 describes the basic shape of the condition development (β_2 *<1=digressive;* β_2 *=1=linear;* β_2 *>1=progressive*). The remaining service life can be calculated at any time, based on the difference between predicted service life and actual service time [Hoffmann, M 2013b].

$$y_a = \beta_0 + \beta_1 * x_a^{\beta_2} \quad \Rightarrow \quad x_c = \left(\frac{y_c - \beta_0}{\beta_1}\right)^{1/\beta_2} \tag{2}$$

$$x_{r,0} = x_a - x_c = x_a - \left(\frac{y_c - \beta_0}{\beta_1}\right)^{1/\beta_2} \tag{3}$$

y_a … failure condition
x_a … average service life
$\beta_{0,1,2}$ … regression factors
y_c … given intervention condition
x_c … average service time to reach a given condition
$x_{r,0}$ … average remaining service time until failure without measure

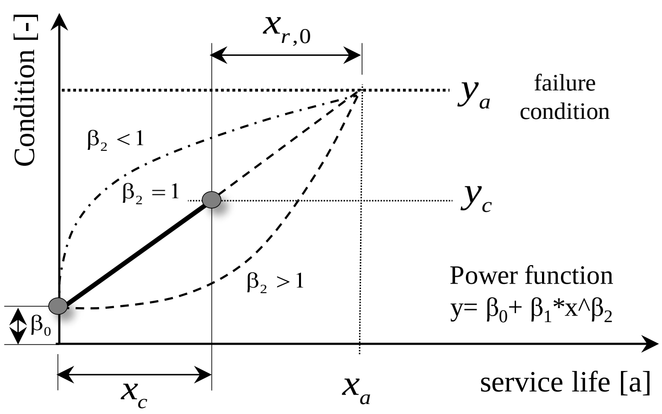

Figure 15: Deterministic model of condition prediction with adjusted "power functions" as master functions [Hoffmann M. 2013b]

2.3.7.1 Probabilistic models

For probabilistic models, it is assumed that both deterioration development and condition survey data, are *subject to uncertainty*. Therefore, these methods provide a *probability* that the respective element is in a *particular condition threshold or fails*. Influencing factors are considered *as random variables* and show a *statistical distribution* [Abra, E. 2012].

Markov chains are considered to be the most popular probabilistic models for describing asset deterioration. Furthermore, logistic regression, multiple discriminant analysis, cohort survival and proportional hazard models are widely used in probabilistic modelling. Markov models outline the probability, p_{ij} that an element in state (*i*) at time-step (*t*), will be in state (*j*) at time-step *(t+1)*. The resulting *transition probabilities* are assembled in the form of a transition matrix (4).

$$p^{t,t+1} = P\left(X_{t+1} = j \mid X_t = i\right) = \begin{bmatrix} p_{11} & \cdots & p_{1j} \\ \vdots & \ddots & \vdots \\ p_{i1} & \cdots & p_{ij} \end{bmatrix} \tag{4}$$

where $p_{ij} \geq 0$; $i,j \geq 1$; $\sum_{k=1}^{j} p_{i,k} = 1$.

The distribution of network states of assets at time *(t+n)* can be found by calculating the product of the *current distribution* and the *transition matrices* according to formula (5). The resulting condition distribution provides information for the total *share of elements* in *any condition class* and time for assets as well as asset networks. The total condition at any given time may thus be calculated from an initial condition distribution. Markov models therefore provide an “a-priori” probability for reaching a certain condition.

$$Q(t+n) = Q(t) \cdot P^{t,t+1} \cdot P^{t+1,t+2} \cdot \ldots \cdot P^{t+n-1n,t+n} \tag{5}$$

In probabilistic infrastructure deterioration modelling, p_{ij} is defined as the probability of an asset deteriorating from (discreet) condition (*i)* to discreet condition (*j)* [Abra, E. 2012].

The predicted condition development based on classic homogenous Markov chains is not accurate either for a number of real life systems due to the simplification of *constant transition probabilities*. To overcome this drawback, a calculation of transition probabilities for different age classes is needed, *requiring condition information* from different age classes that are not available in most cases.

While future states are dependent only on the present state in a *time-homogenous* Markov model, independently distributed random variables are used to model the time between condition states in a *semi-Markov* or *non-homogeneous model*. As for asset deterioration, this means that deterioration probability increases with element age. A disadvantage of simple Markov chains is that the deterioration process is described inadequately, especially at the beginning and end of life time, which decreases prediction accuracy [Hoffmann, M. et al. 2014e].

Deterministic performance predictions show steady development for all elements in the same age class from excellent condition to a state of failure all at the same time. However, in most cases this is not true due to the *stochastic distribution* of service lives *making deterministic models* a very *rough approach* [Hoffmann, M. 2013 c]. Despite common probabilistic and deterministic deterioration models, *soft computing methods*, such as *artificial intelligence*, are often applied to describe asset condition. These models are able to model unknown, complex and nonlinear relationships between inputs and outputs based on a few underlying assumptions, even allowing the use of imprecise, incomplete and subjective data. However, the large amount of data needed for *training* and *calibration* of the model, along with a non-transparent path to the solution (*black-box*), have to be considered as major setbacks of these methods, which consequently result in a niche existence [Abra , E. 2012].

The selection of an appropriate method for condition modelling and prediction is of high importance as method selection further influences the *significance* and *validity* of different *risk considerations*. If considerations of failure risk are an important criterion for infrastructure operators or owners, then probabilistic approaches should be applied preferably, since deterministic approaches allow only restricted statements about failure risk as they do not take into account the statistical distribution of condition data. Thus it is hardly possible to attribute risk in a mathematically correct manner with common approaches relying on an attribution of risk categories and extents to condition grades or certain thresholds. Contemporary deterioration models therefore mostly use probabilistic approaches on an empirical basis.

2.3.8 Collection of condition data and adaption of master curves

Condition survey, also referred to as *structural health monitoring* (SHM)[16] or *condition detection*, constitutes the foundation for any *empirical condition* modelling approach in asset management and the basis for any investment decision as well. The foundation for any successful detection and storage of element condition is defined already with the conception of the *core database*. While deterioration on element level is described by *master curves* (common mathematical functions are shown in Figure 13) adjusted to the actual condition development by standard techniques for calibration as *scaling*, *vertical* & *horizontal shifting* and *regression* (see Figure 17), average condition developments as master curves with failure distributions yielding a sum of assets in different condition at a time are applied on route level (Figure 16).

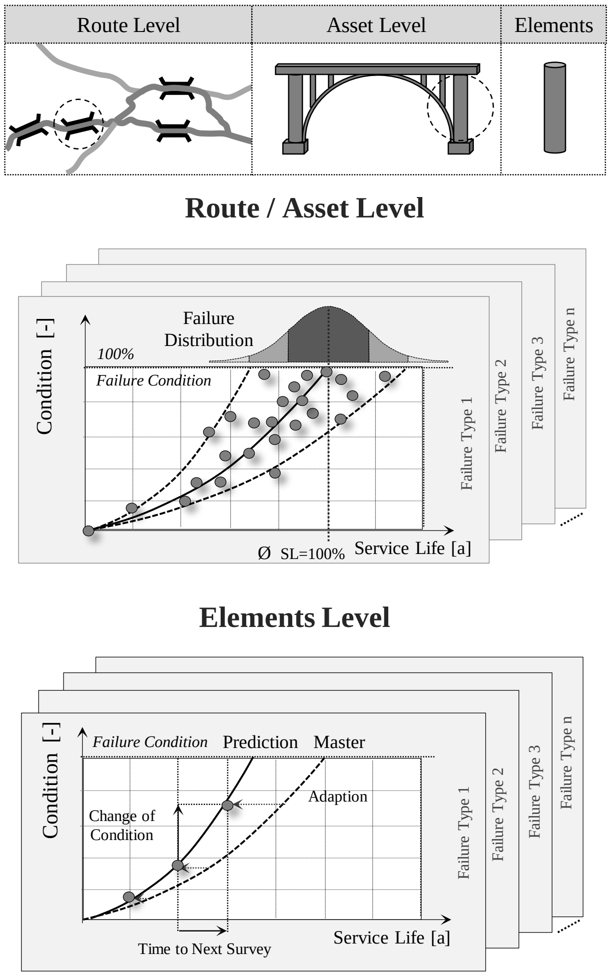

Figure 16: Average condition development and schematic failure distribution on route level; on elements level average master curves for different failure types have to be adjusted to the actual condition development.

If only *one condition survey* is available and the age of the element unknown, the adaption of the master curve may be performed by *vertica*l or *horizontal shifting*. In order to apply the method of *curve scaling*, either *two condition detections* have to be available, or actual *age* and *initial condition* of the element have to be known in addition to *one available condition* survey [Hoffmann, M. 2015b]. With a number of condition measurements at hand, master curves can be customized by *regression*. Thereby, the basic shape of the curve (mathematical function) may either be known a priori, or all parameters are chosen freely (*best fit*). Mathematically, a best-fit function describes an equation that passes through measured points so that the sum of squared deviations of data points from the regression function is minimized (e.g. linear regression between one dependent and one independent variable).

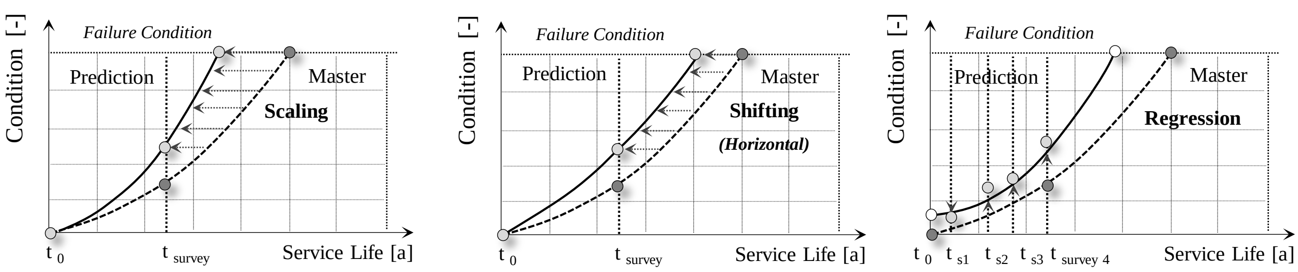

Figure 17: Adaption of master curves to measures condition data based on scaling, shifting and regression [Donev, V. 2014]

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[16] *Structural health monitoring describes the process of implementing a damage detection and characterization strategy for engineering structures.*

For models with more than one *explanatory variable* and a *nonlinear best-fit function*, the derivation of the parameters can be *exceedingly complex*, resulting in the need for specialized statistical software. In order to evaluate how well a model explains the condition data, a number of tests can be accomplished, with the *coefficient of determination*[17] (R^2) being most commonly used. The validity of the model is then adjudged to how closely it fits to empirical observations, and how well it extrapolates to data. The requirements for the selection of a mathematical model are a good match with the actual behavior and the possibility to include all main influence factors. With any additional condition survey, a more accurate calibration of the selected deterioration model is possible [Labi, S. 2014].

Within the scope of condition survey, a number of *failure modes* with different *shapes of failure progress* are detected for individual asset elements. Consequently, both, *failure extent* and *failure severity*, as basic parameters of the monitoring concept are collected. Thereby, the detected failure severity subsequently indicates both the *basic need for measures* and the appropriate *measure type*. The detection of failure extent, however, is crucial for the *derivation of required measure extent*. In order to ensure a consistent and systematic condition survey, the definition of all *significant failure types* together with *typical damage patterns* and the *underlying deterioration model* is required beforehand, illustrated by *failure catalogs.* Condition detection and modelling on elements-level is followed by bottom-up aggregation of condition development to assets and routes or networks [Hoffmann, M. et al. 2012 a].

Condition monitoring is often performed as a part of *larger survey campaigns* in periodic intervals. The *prediction accuracy* of the deterioration model increases with the *number of detections*. Thus, with each condition survey a further data point for the adaption of master curves is available, in line with the basic idea of *self-learning* asset management systems.

Furthermore, the selection of inspection method (visual or automated data collection techniques) depends on the accessibility of the asset element, failure characteristics and the necessary monitoring effort. The frequency of surveys, necessary for a reliable condition prediction, depends on a number of parameters such as *remaining time until failure*, *extent* and *severity* of failure consequences as well as *respective network size*. Routine condition surveys usually constitute the basis for the development of budget scenarios, treatment strategies and priorities, whereas detailed surveys serve as a foundation for *specific measure planning*, comparison of maintenance alternatives, and *quality assessment* after implementation.

In complex systems with a large longitudinal extension, condition surveys are often supported by *GPS-positioning* with additional characterization of element condition by *photo-documentation*. Essential for the characterization of condition development are both the time period between two consecutive surveys and the change of condition during this time period (Figure 16).

Since the *inclination of the deterioration curve* significantly influences element service life, and thus, the life cycle costs of infrastructure assets, a *statistical validation* of the selected deterioration model is as important as accurate structural condition monitoring itself.

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[17] *The coefficient of determination R^2 is measure used in statistical model analysis to assess how well a model explains and predicts future outcomes. It is indicative of the level of explained variability in the model. The coefficient is used as a guideline to measure the accuracy of the model. (see also http://www.investopedia.com/terms/c/coefficient-of-determination.asp)*

2.3.9 Condition assessment and service life

Measured condition values by themselves, obtained from condition surveys, do not provide information on whether the condition of an element is *relatively good* or *relatively bad*, or *whether maintenance measures* should be applied. Common *rating approaches* in condition assessment are mainly based on weighted scores, allowing a relative comparison of different assets with their extent and severity. Typical *condition grades* range from 1 to 5 (1=very good; 5= failed) and are color coded, making them easy to read.

Performance functions as rated condition functions provide a good *overview* of infrastructure condition on network or route level, and allow the identification of assets and asset elements with a *higher priority* in measure implementation. The *transformation* of measured conditions is followed by a *weighted aggregation* towards overall grades. Within the transformation process, measured condition values with their respective units are converted into *dimensionless condition indices* by using deterioration-specific *rating functions* or *condition thresholds*. The application of *weighting factors* for different deteriorations modes allows for the inclusion of user aspects, such as *safety* and *comfort*, separately from structural condition or the calculation of one overall condition index.

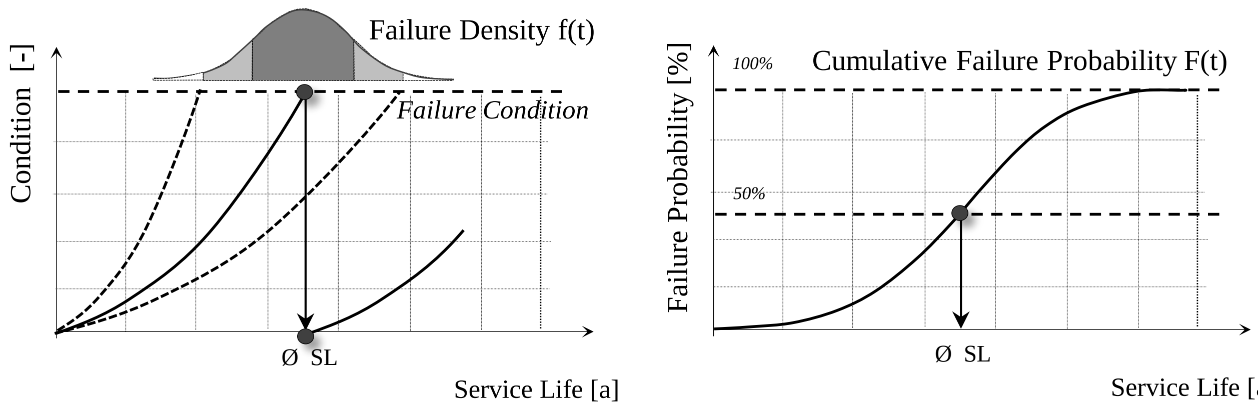

Figure 18: Probabilistic failure assessment and average service life; own compilation based on [Hoffmann, M. 2009].

Typically, infrastructure asset management approaches use the *lower limit* of the *worst condition class* as a decisive failure condition (*failure threshold*). Taking into account deterministic model assumptions would mean, that a critical system would fail at an exact date, making a simple replacement the day before very cost effective due to a *full exploitation* of *theoretical service life*. Since the probability of a system failing exactly on this day is almost negligible, inaccuracies in condition prediction are very likely resulting in severe failure impacts with *high follow-up costs,* both for *infrastructure operators* and *users*. In order to avoid an actual failure condition, appropriate measures must thus be implemented usually prior to a deterministic predicted end of service life. Therefore, the costs to mitigate worse element condition are increasing continuously, with increasing age. The optimization of intervention time is based on the lowest annuity and is backwards oriented, starting with the potential failure time.

For infrastructure assets, there is always a distinction between *structural failures* and *functional failures*. Structural damages may have a lower (e.g. pavements) or higher (e.g. bridges) impact on user safety and infrastructure reliability than functional damages. However, deterioration exceeding certain condition thresholds for structural damage types indicates that cost-intensive renovations are required.

In common asset management approaches, unsystematic *threshold-values* are defined which subsequently *trigger* the implementation of required measures or an *intensification* of monitoring intervals. The derivation of these threshold-values, however, may be based on *minimum standards*, *condition distributions*, *safety considerations*, or *expert opinions*. For elements with a known condition development, defined failure conditions automatically determine the average *service life* of an element. This service life is directly related to condition threshold-values (Figure 18).

Condition thresholds prior to actual failure allowing worse conditions, subsequently lead to a fuller utilization of service life at the cost of increased failure probability (Figure 19). Thus, the establishment of threshold-values results in a restriction of *average asset service life* (deterministic) with a certain variance of service life for different confidence levels (probabilistic). Fixed failure thresholds for *non-safety* related failure types should therefore be avoided, meaning that the determination of optimal intervention time becomes *a purely economic task*. If element failures affect the users of an infrastructure, user costs should be considered in addition to minimum legal standards for the establishment of failure thresholds. The determination of these threshold-values should therefore be handled with special care.

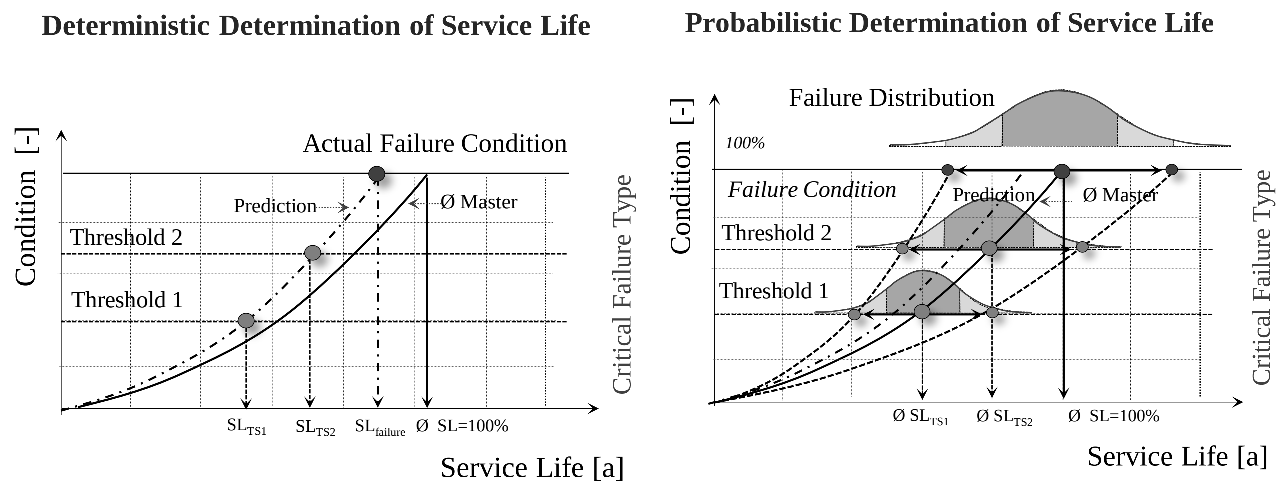

Figure 19: Service life being directly related to condition threshold-values with increasing thresholds leading to increased average service life (deterministic approach) and increased average service life and variance of failure distributions (probabilistic approach); own compilation based on [Hoffmann, M. 2009].

In practice, condition threshold-values for non-safety related deterioration modes are often established based *on expert opinion* as an acceptable *first assumption*. In times of limited budgets, condition thresholds should be verified statistically and take tolerable failure probabilities into account. For deterioration modes, which, however, do not affect customer safety or service comfort, the determination of thresholds in infrastructure asset management is a purely economic problem for infrastructure operators [Hoffmann, M. 2015].

2.3.10 Failure impact and optimum of total costs

Since no infrastructure system provides a *zero rate of failure*, and no human activity can be performed completely free of risk, safety approaches aiming at *risk optimization* have been developed. These approaches attempt to balance the *risk* of a given activity (e.g. *infrastructure use*) against its *benefits,* and seek to assess the need for further risk reduction depending upon the costs (compare Figure 20) and benefits. Similarly, reliability engineering compares costs of reducing failure rates against the value of the enhanced performance [Smith, D. 2011].

Infrastructure investments are described as *capital-intensive* and *long-term* oriented. Thus, the results of erroneous decisions, often become visible only *decades later*, and are then associated with significant *financial efforts*. The aim of infrastructure asset management is, therefore, to ensure *economically* and *technically* optimized development infrastructure assets and maintenance activities. When a failure condition occurs, *follow-up costs* arise for infrastructure operators (*reconstruction*, and *liability*) and users (*accidents*, *detour*, and *congestion*).

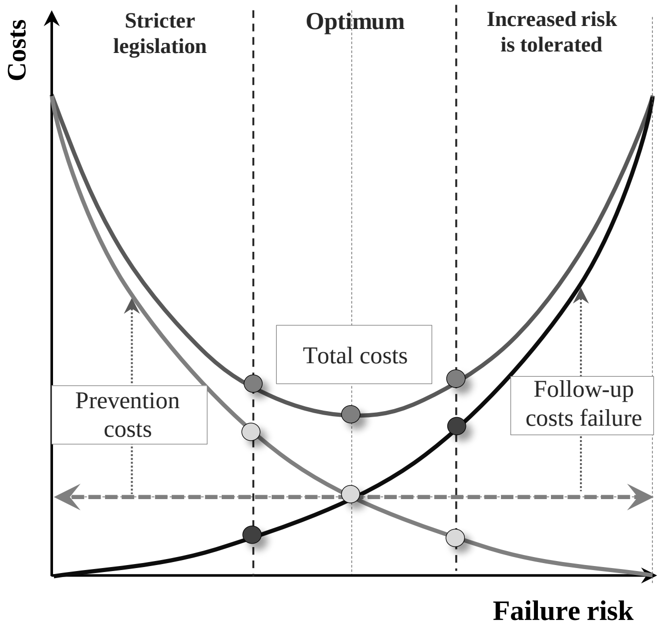

Figure 20: Derivation of the cost optimum at a minimum of total costs for users and maintenance costs [Balzer, G. & Schorn, C. 2011]

Legal regulations often specify that infrastructures should be constructed and operated such that they offer a *safe* and *reliable* operation process, but also account for *economic efficiency*. That means, technical framework conditions, such as infrastructure *quality* or infrastructure *condition*, and economic framework conditions, such as *measure efficiency* and *low annuity*, must be evaluated against each other as illustrated in Figure 20. If condition deterioration exceeds the defined failure threshold, poor infrastructure quality results in *follow-up costs* for operators, ranging from premature asset renewal, up to the payment of liability costs in case of failures or accidents. Infrastructure operators influence infrastructure quality by the implementation of proper maintenance and rehabilitation measures, preventing failure, and are therefore often named *prevention costs* [Balzer, G. & Schorn, C. 2011].

With increasing investments in maintenance activities, and thus asset condition, infrastructure safety and reliability increase as well, resulting in a reduction of follow-up costs for infrastructure operators and users. Eventually, however, investments thereby exceed the point where *economic efficiency*, based on life cycle considerations, is no *longer attainable*. High *prevention costs* are only accepted, if financing of such infrastructure quality standards is economically affordable and intended by the society, and the necessary legal requirements are given as well. In practice, the willingness to accept an outage of assets exceeds the willingness to pay for avoiding failures. If, however, little or nothing at all is invested in infrastructure maintenance, asset deterioration progressively increases with subsequently increasing follow-up cost for users and operators. Such asset management strategies can only be applied, if infrastructures, with higher quality standards, cannot be applied, due to *financial setbacks* and a *society* that *tolerates* high follow-up costs (strategy of externalization of costs).

The optimal infrastructure quality from a *technical* and an *economic point of view*, can be found at a point where the *total cost* (sum of maintenance and follow-up cost) become a *minimum*. If maintenance strategies are compared to each other, the strategy with the lowest total costs should be preferred. The main task of infrastructure asset management is, therefore, to derive an *optimized infrastructure quality*, based on the methods of economical assessment and life cycle cost analysis, within the boundaries of legal requirements.

2.3.11 Measure impact and impact duration

In order to compare maintenance measures of individual assets or asset elements with each other, the condition model must include a prediction of the deterioration process with measures and respective measure costs. To model the *impact of a measure* on the further condition development, the *condition reset value* y_d as well as the *duration* of *measure impact* x_b must be known. The resulting deterministic condition development may be based on power functions can be calculated according to equations (6) to (11) for maintenance, rehabilitation or renewal (see Figure 21) [Hoffmann, M. 2013b].

x_c … intervention time
y_c … given intervention condition
y_d … estimated condition reset value = measure impact
x_b … estimated duration of measure impact
$x_{r,m}$ … remaining service life with measure
$\beta_{0,1,2}$ … failure propagation factors with a measure for given x_b and y_d
x_d, x_e … auxiliary variables

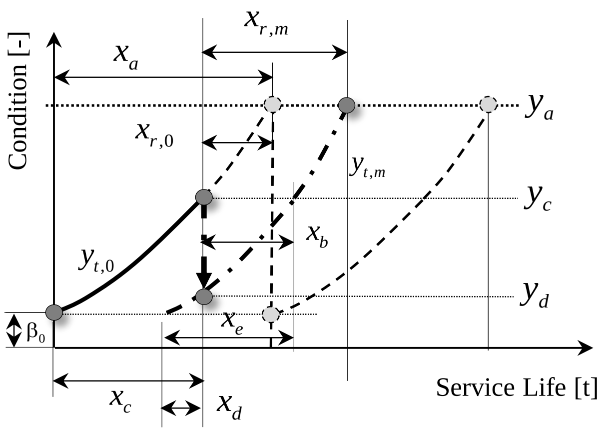

Figure 21: Deterministic model of condition performance prediction with perpetual reconstruction or rehabilitation based on their impact and service time [Hoffmann M. 2013b]

$$x_e = x_d + R_d = \left(\frac{y_c - \beta_0}{\beta_1 * \beta_3}\right)^{1/\beta_2} \qquad x_d = \left(\frac{y_d - \beta_0}{\beta_1 * \beta_3}\right)^{1/\beta_2} \tag{6}$$

$$x_b = \left(\frac{y_c - \beta_0}{\beta_1 * \beta_3}\right)^{1/\beta_2} - \left(\frac{y_d - \beta_0}{\beta_1 * \beta_3}\right)^{1/\beta_2} \tag{7}$$

$$x_{r,m} = \left(\frac{y_a - \beta_0}{\beta_1 * \beta_3}\right)^{1/\beta_2} - \left(\frac{y_d - \beta_0}{\beta_1 * \beta_3}\right)^{1/\beta_2} \tag{8}$$

$$\beta_3 = \left(\frac{(y_c - \beta_0)^{1/\beta_2} - (y_d - \beta_0)^{1/\beta_2}}{x_b * \beta_1^{1/\beta_2}}\right)^{\beta_2} \tag{9}$$

$$y_{t,0} = \beta_0 + \beta_1 * x^{\beta_2} \qquad \text{for } x_t \le x_c \tag{10}$$

$$y_{t,m} = \beta_0 + \beta_3 \beta_1 * (x_t - x_c + x_d)^{\beta 2} \qquad \text{for } x_t > x_c \tag{11}$$

Actual *impact duration* of rehabilitation measures and *service life* of infrastructures are derived from laboratory tests, estimated as "*best expert-guess*", or determined backwards with an increasing amount of data based on the asset management database.

Depending on the progress of different failure types and their combination, different measures or combination of measures may be applied. *Measure* versus *failure type matrices* allows for a preselection of feasible, applicable measures for the optimization process. An integral part of this matrix are *condition reset values* of single measures on the respective failure types. Usually, a *full condition reset* to the *initial asset condition* is often assumed as a first estimation. For a more comprehensive model, the measure impact on each failure type has to be described in detail. This approach is considered as data-intensive and therefore often based on *expert guess* or *best estimates* and later on adjusted to *empiric parameters* from the feedback of implemented results [Hoffmann, M. 2006]. If the condition of an infrastructure asset is improved by a rehabilitation measure, the deterioration model after measure implantation may be different compared to the initial deterioration process of the respective failure type.

The deterioration curve after measure implementation depends, of course, on parameters such as the *failure type*, *failure severity* and *measure type*. Figure 22 illustrates that the deterioration process after measure implementation may either *remain the same* compared to the initial condition development, *change significantly*, or show a *deviation* compared to the initial condition *after a certain period* (increased failure progressivity). If a failure condition is fully repaired by a rehabilitation or renewal measure, the further condition development can be shifted with its origin by the value of condition reset (new master curve). If a rehabilitation measure improves only the actual condition value but does not change the deterioration trend, the curve of condition development after measure implementation shows a parallel course to the initial deterioration (parallel master curve). Consequently, the function of condition development is shifted vertically downwards. If an improper measure is applied, or a measure is not implemented correctly, the deterioration process either remains unaffected (no measure impact) or proceeds with increased progressivity (temporary condition improvement) [Weninger Vycudil, A. 2001].

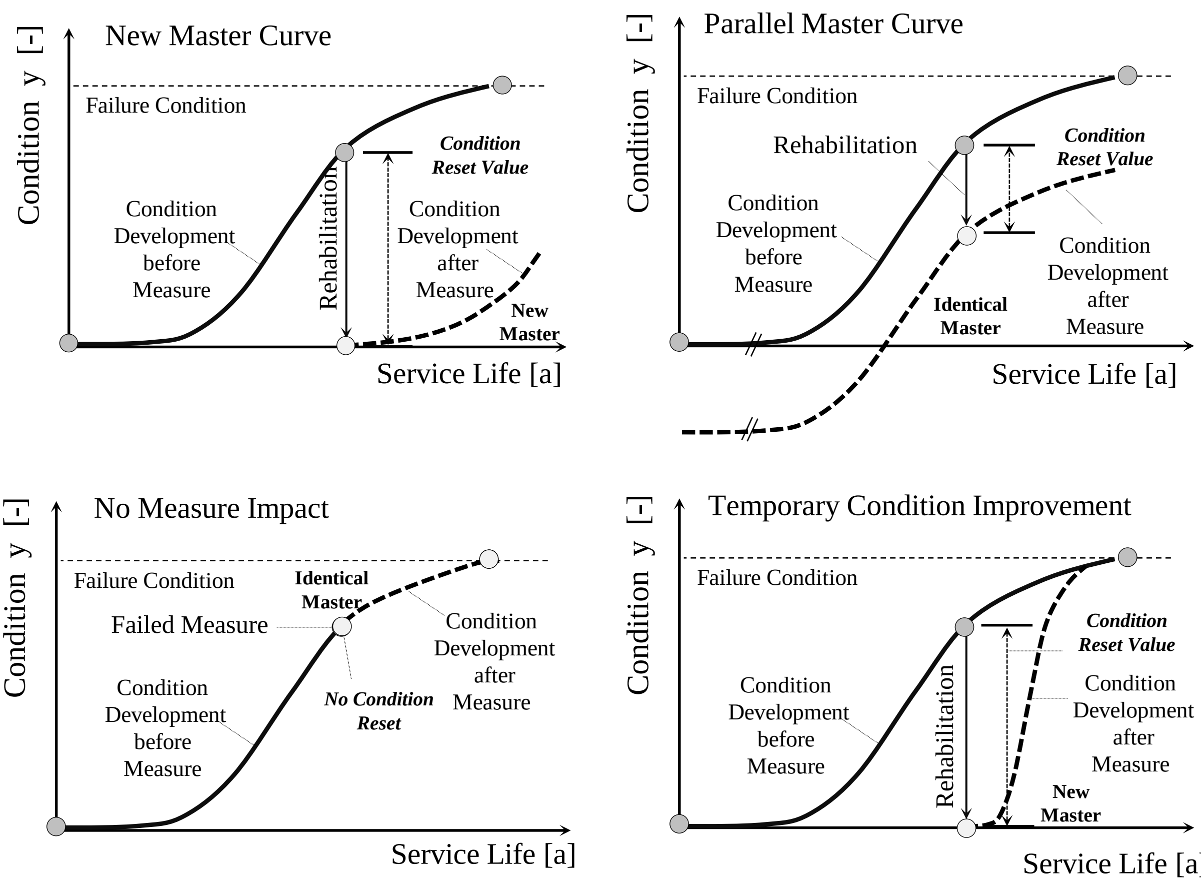

Figure 22: Schematic illustration of possible impacts of rehabilitation measures on the deterministic condition development [Weninger Vycudil, A. 2001]

2.3.12 Fundamentals of investment appraisal

Investment decisions in infrastructures asset management are *forward-looking* decisions about the use of available financial resources (e.g. for measure selection) and thus have a considerable impact on the *economic development* of individual infrastructure assets as well as on the managers of infrastructures (owners, operators). Infrastructure owners may provide the financial resources for the construction and preservation of their infrastructures, either by *equity* or *debt capital* as *internal* or *external* financing. In terms of timeframe, there is a major distinction made between *short- and long-term* oriented financing. For the comparison of investment alternatives in infrastructure asset management the entire life cycle of assets is often defined as an observation period. The incurred costs are also referred to as *life-cycle costs* or "*cradle to grave*" costs. For the creation of a life-cycle cost model it is therefore essential to identify all costs that have a significant impact on the cost development during the life cycle. Life-cycle cost models may either remain restricted to operator costs or take user costs and external costs into account. Typical expenditures for operators include planning, design, construction, operation, maintenance and renewal costs (see Figure 24). Maintenance expenditures may account for many times the amount of the initial cost of the asset. Thus, life-cycle considerations are especially important for assets with a long service life. In the view of a market-oriented private infrastructure operator, the maximization of financial assets of the shareholders constitutes the main goal in the respective asset management strategy (compare chapter 2.3.4). [Fischer, E.O. 2002].

As with typical and conventional methods of investment appraisal, *static methods* which do not include both interest rate and investment timing, or *dynamic methods* which account for both and methods of *risk analysis*, are applied. The static methods include criteria such as the average profit and the static payback period. Dynamic methods include *net present value* (NPV), *equivalent annual cost* (EAC), *internal rate of return* (IRR) and *dynamic payback period*. Static methods are easy to handle, however due to the fact that neither real incurred capital costs nor the timing and order of payments are taken into account, their application for investments optimizations within the life cycle is limited [Fischer E. O. 2002]. Therefore, net present value and the comparison of investments based on annuities will be described more in detail since they are used for the economic assessment of investment options in this thesis.

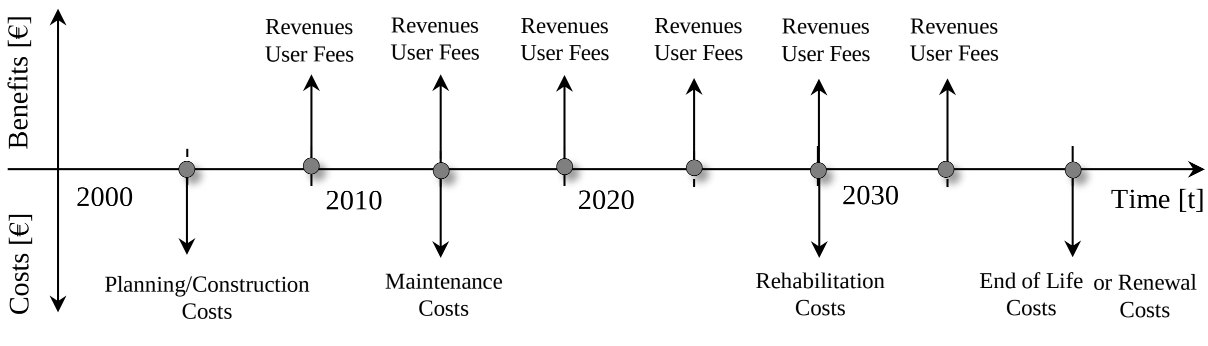

Figure 23: Visualization of typical inflows and outflows of cash as foundation for economic assessment

As most important input parameters for the economic assessment of any infrastructure, investments have to cover all relevant *inflows* and *outflows* of cash (see e.g. Figure 23); the respective *timing* of the investment, and the *expected service life* or *impact duration* as well as the *interest rate*. This interest rate substantially depends on the financing method of the project. While, in the case of equity financing, the value of the average rate of return of an alternative investment, determines the interest rate, the effective interest rate of the debt capital has to be applied for the method of external financing.

The interest rate constitutes a time compensating function, at first, as with its help investments with a different investment-timing can be discounted to a reference date. At the same time, the interest rate reflects the expectations of the investor regarding the minimum interest rate that should be achieved for the capital. Depending on the method of financing, average interest rates of approximately 2% up to 4% are applied for the comparison of infrastructure investments and subsequent sensitivity analyzes.

2.3.13 Cost elements in asset management

Within the realm of infrastructure asset management, *costs* are considered as *valued quantitative* consumption of goods (or assets) for the creation of constructions and services[18]. For the purposes of infrastructure asset management, cost estimations for the evaluation of infrastructure constructions and measures may include *owner*- and *operator costs* (construction costs, operation costs, maintenance costs and renewal costs), *user costs* (travelling costs and vehicle operation costs) and *external costs* (noise, air pollution, energy and follow-up costs of accidents) according to Figure 24, with owner- and operator costs as the main focus of this thesis.

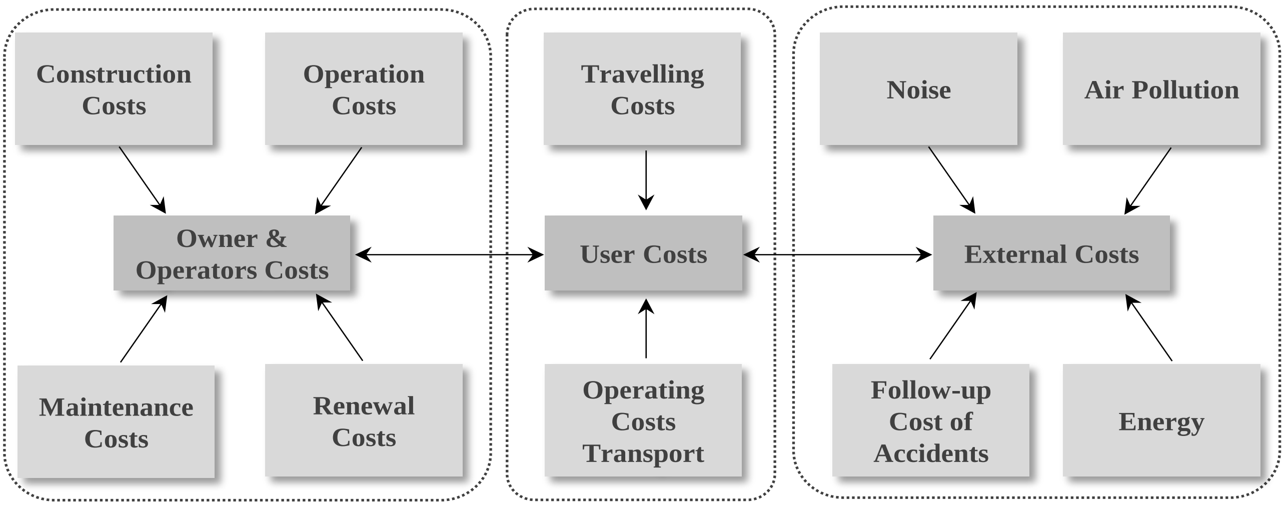

Figure 24: Cost elements of transport infrastructures for owners and operators, users and third parties; own illustration based on [Blab, R. 2014]

Cost estimations in infrastructure asset management serve different purposes. Thus, a calculation of total costs for individual business areas provides the basis for the derivation of general performance indicators. As an example, the costs of infrastructure operation may be divided by network kilometers and thus represent the basis for the comparison of general business performance with other infrastructure providers.

The estimation of total costs for individual asset categories (e.g. bridges and tunnels) and lifecycle phases (planning, construction, and operation) offers important key figures for further costs estimations (e.g. new construction projects). The determination of total costs of asset elements with both a homogeneous function and service life, such as bridge foundations, allows for more informed life cycle calculations. The cost analysis of service items, however, serves to ensure systematic evaluation of fluctuation ranges of price offers, and facilitates tendering as well as the award of contracts.

Cost estimations in infrastructure asset management are based upon *cost models*. These cost models usually include estimations for the costs of *materials*, *machine time*, *labor costs* and *additional effort*, which

[18] *RVS 02.01.14 Determination of the Costs of Infrastructure Projects (2012).*

is necessary to implement construction and rehabilitation measures. For the accuracy of estimations of the future costs development, a thorough data analysis is essential.

Depending on calculation objectives and available data sets, cost estimations for measures can be implemented either *top-down* (deduced from top down) or *bottom-up* (costs are estimated aggregating from the bottom up)[19]. Top-down approaches estimate the total costs of a construction measure in a first step. Thereby, measures are subdivided in *typical working steps* and link them with typical service items, which are related to unit cost functions (illustrated in Figure 25) derived from already implemented projects.

In a second step, estimated total costs are separated down to the single service positions. This requires that the shares of either working steps or service items are known. By contrast, bottom-up approaches estimate the cost for typical components, and subsequently multiply unit costs and quantities.

Standard input parameters of costs models for measures in infrastructure asset management include *main cost components*, with their *unit costs* (total costs divided through quantity), and the intended *measure extent*. As main output, these models provide estimations of total measure costs. In practice, these estimates are often conducted backwards based on operational accounting from previous periods and already implemented projects. Depending on available data, these models can also be extended to parameters that account for the framework conditions.

Figure 25 illustrates the basic relationship between *unit costs* and *measure extent* as well as the development of *total costs*. As an example, a regressive unit cost function and a declining function of total costs are illustrated because they apply best to economy of scale for construction measures of the same type. Thereby, for all measures occur, both *fixed costs*, such as overhead and cost for construction site facilities, and *variable costs*, which rise with increasing working hours and material expenditures. The costs functions (flexible power functions), which are used in the presented approach, will be further described within chapter 2.3.14.

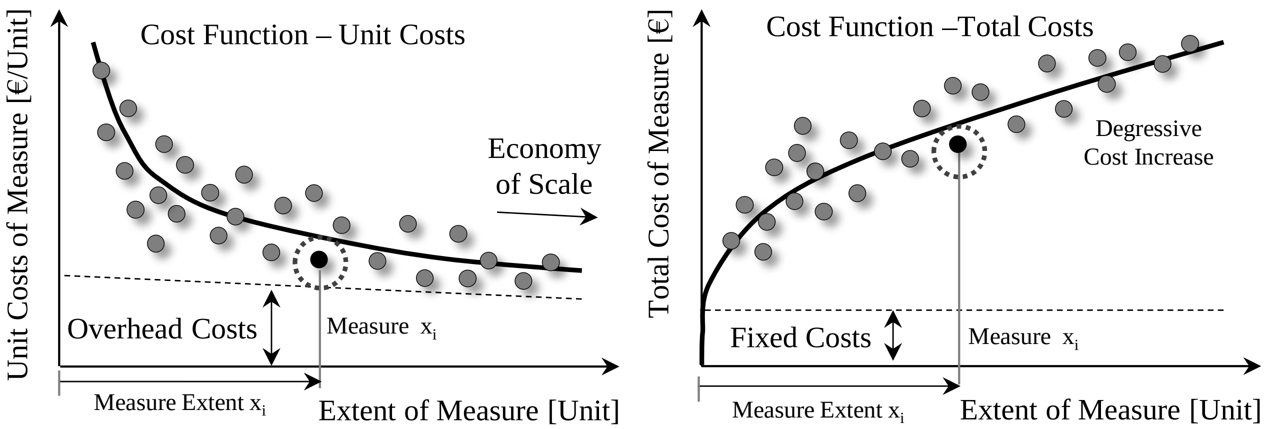

Figure 25: The cost model for construction and rehabilitation measures of infrastructures describes the development of unit cost and total measures costs with increasing measure extent (e.g. decreasing unit cost and digressively increasing total costs).

[19] http://www.wirtschaftslexikon24.com/d/bottom-up-top-down-planung/bottom-up-top-down-planung.htm; *(Last access: 30.04.2015)*

In practice, various cost functions are used to describe the development of total costs, taking into account both, fixed cost (β_4) (offset-value) and the cost increase with increasing measure extent (x).

Applicable functions for total costs (C_{total}) and unit cost (C_{unit}) differ regarding their general shape, and may be divided in linear as well as non-linear types (Figure 26)[20].

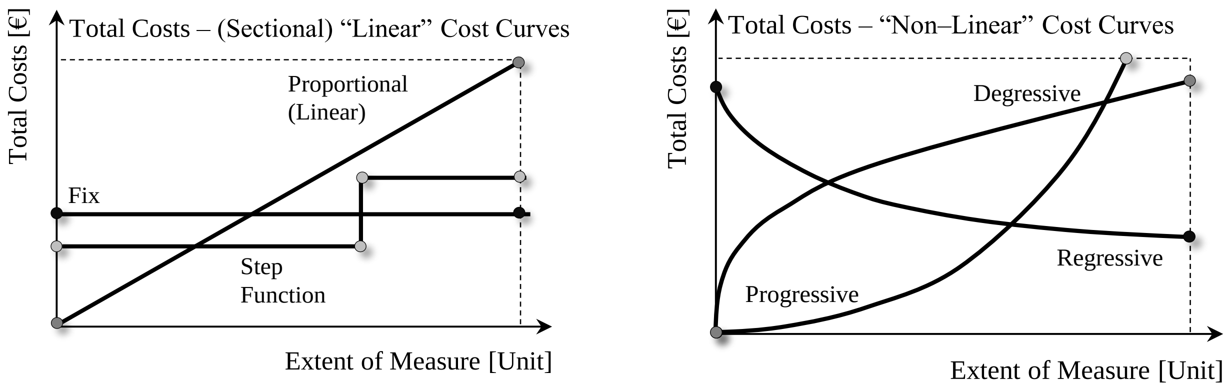

Figure 26: To determine total costs of infrastructure measures, a number of linear and non-linear costs functions, as illustrated above, can be used.

Furthermore, equations (12) to (23) provide an overview of the calculation of total and unit costs using linear and non-linear cost functions, with β_5 and β_6, being regression parameters [Samuelson, P. A. et al. 1998].

1) **Fixed costs**

For fixed cost functions, the total costs of a measure remain constant, independent of the extent of a measure. Fixed total costs are very common in the construction industry, when "*all-inclusive prices*" are established for the implementation of certain construction works, without any proof of actually executed measure extent.

$$C_{total} = \beta_4 \qquad (12)$$

$$C_{unit}(x) = \frac{\beta_4}{x} \qquad (13)$$

2) **Proportional cost function**

For linear cost functions, variable costs and subsequently total costs increase in the *same proportion* as the measure extent. Consequently, unit costs remain constant, irrespective of the measure extent.

$$C_{total}(x) = \beta_4 \cdot x \qquad (14)$$

$$C_{unit} = \beta_4 \qquad (15)$$

3) **Step cost function**

Total costs *remain constant* for *certain intervals* of measure extent. Between these intervals, costs "*jump*" to another, higher level. The resulting function of total costs shows a *step-like* development. Such cost functions, in practice arise when a fixed "*all-inclusive price*" is agreed upon, up to a certain threshold in measure extent. Thus, for example, costs for the transport of construction material may increase step-wise when a certain transport radius is exceeded.

[20] *Also see https://de.wikipedia.org/wiki/Kostenfunktion_%28Wirtschaft%29; [Last access: 30.04.2015]*

$$C_{total}(x) = \begin{Bmatrix} \beta_6,\ x < \beta_4 \\ \beta_5,\ \beta_4 \le x \le \beta_6 \\ \ldots,\ \ldots \end{Bmatrix} \quad (16) \qquad C_{unit}(x) = \begin{Bmatrix} \frac{\beta_6}{E},\ x < \beta_4 \\ \frac{\beta_5}{x}, \beta_4 \le x \le \beta_6 \\ \ldots,\ \ldots \end{Bmatrix} \quad (17)$$

4) **Regressive cost function**

In this particular case, *total costs,* as well as their *unit costs, decrease* with an *increasing value* of the *reference variable* (e.g. heating costs in construction trailers with a rising number of employees or visitors). This type of cost function is rarely used in practice.

$$C_{total}(x) = \beta_4 \cdot x^{-\beta_6} \quad (18) \qquad C_{unit}(x) = \frac{\left(\beta_4 \cdot x^{-\beta_6}\right)}{x} \quad (19)$$

5) **Progressive cost function**

A progressive cost function is characterized by the fact that with *increasing measure extent, variable costs* increase *disproportionately*. Moreover, unit costs also rise. As an example, progressive cost functions appear for measures which address *highly progressive failure types,* or measure with a *progressive damage potential* for the environment and *capacity bottlenecks*, may be mentioned.

$$C_{total}(x) = \beta_4 \cdot x^{\beta_6} \quad (20) \qquad C_{unit}(x) = \frac{\left(\beta_4 \cdot x^{\beta_6}\right)}{x} \quad (21)$$

6) **Digressive cost function**

A declining cost function is characterized by the fact that with increasing measure extent, *variable costs increase less than proportionately*. Unit costs thus decrease with increasing measure extent (e.g. due to granted discounts for high quantities).

$$C_{total}(x) = x^{\frac{\beta_4}{\beta_6}} \quad (22) \qquad C_{unit}(x) = \frac{\left(x^{\frac{\beta_4}{\beta_6}}\right)}{x} \quad (23)$$

Inherently, input parameters of cost models are subject to certain variances. However, the results of cost estimations become *more stable*, when costs are *aggregated* up to *larger networks*. The *significance of such estimations decreases,* however, as statements about any cost drivers are not possible. The description of standard types of cost curves already includes some indications on parameters that affect the development of measure costs.

Furthermore, the *general market situation* may be mentioned as a standard impact parameter on resulting measure cost. Since the *ratio of supply to demand* is *decisive* for *pricing* and *resulting costs*, shifts towards an increase in demand may lead to substantial cost increases of measures that have to be addressed by comprehensive cost approaches. Further influences on the level of costs, are increased requirements regarding asset or *measure quality*, a *low risk tolerance* and the general surrounding area of an infrastructure. Thus, construction and maintenance of infrastructure assets, especially in urban areas with a high density of buildings, may be described as significantly more expensive. The same applies for sensitive areas around protected natural areas.

However, economy of scale takes the most important role in cost estimations for infrastructures. In order to realize reductions in unit costs, current trends head towards increasing lengths of constructions sites. Especially when expensive machinery is used for small construction sites, expensive fixed costs significantly increase both unit- and total costs.

2.3.14 Life-cycle cost analysis and comparison of infrastructure investments

The methods of life-cycle cost analysis are well known by the majority of decision-makers due to the large number of publications and their incorporation in national and international laws. Within the scope of *life-cycle cost analysis*, all relevant events (e.g. construction, rehabilitation), as part of an infrastructure maintenance strategy, are monetized in their life cycle as criteria and indicators for the comparison of net present values and annual costs. The method of life-cycle cost analysis forms not only the basis for the economic comparison of individual measures, but also serves as a foundation for the optimization of investments. The optimization of investments aims at the selection of the economically most efficient measure (mathematical minimization of measure costs as objective function within the infrastructure life cycle). An actual optimization process is much more complex than a simple cost comparison as all conceivable variants (variation of measure type, extent and timing) have to be calculated in order to identify the minimum costs. Depending on the optimization goal, further objective functions, for example user costs, have to be included for a comprehensive investment optimization (compare chapter 2.3.15 and 2.3.16).

The net present value (NPV) in finance is defined as the sum of the *present values* of *incoming* and *outgoing cash flows* during the life cycle of infrastructure assets. The calculation is based on discounting of *future cash-flows* (also taking into account timing of monetization) with *subsequent summation*. As a basic principle of *dynamic investment appraisal*, the time value of money dictates that time has an impact on the value of cash flows. For the implementation of infrastructure maintenance measures, this means that investments in maintenance measures show a *decreasing impact* on the net present value, the later they occur. The net present value can be calculated according to Figure 27 with equation (24) for periodic and running payments.

NPV … net present value
C_p … costs of periodic measures
C_r … running costs per year
i … interest rate (2-4%) with $q = 1 + i$
n … credit runtime (∅ service life)

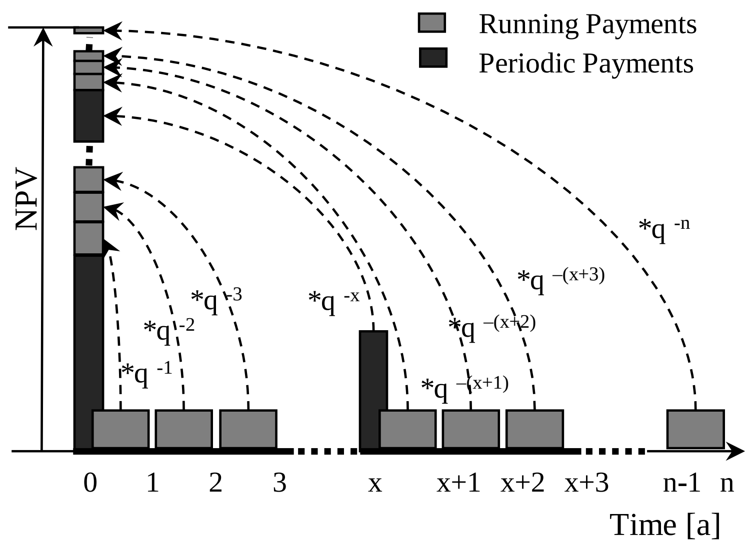

Figure 27: Calculation principle of net present value during a standard life cycle of infrastructure assets [Kruschwitz, L. 2009]

$$NPV = \sum_{t=0}^{n} C_p * q^{-t} + C_r * q^{-t} * (q^t - 1) / (q - 1) \tag{24}$$

Annuity is described as a series of payments at fixed intervals of time. In infrastructure asset management, the *equivalent annual cost* is the cost per year of owning and operating an infrastructure asset over its entire service life. The annual costs are based on the net present value of the life cycle according to Figure 28 through equation (25). The *total repayment* during the service life of infrastructure assets may be obtained through equation (27). Depending on its sign (+/-), the annuity can be considered a *steady amortization rate* of the *negative net present value*, or as *steady revenue of* a positive net present value of an investment.

NPV … net present value
R … repayment rate
TPR … total repayment
i … interest rate (2-4%) with $q = 1 + i$
n … credit runtime (∅ service life)

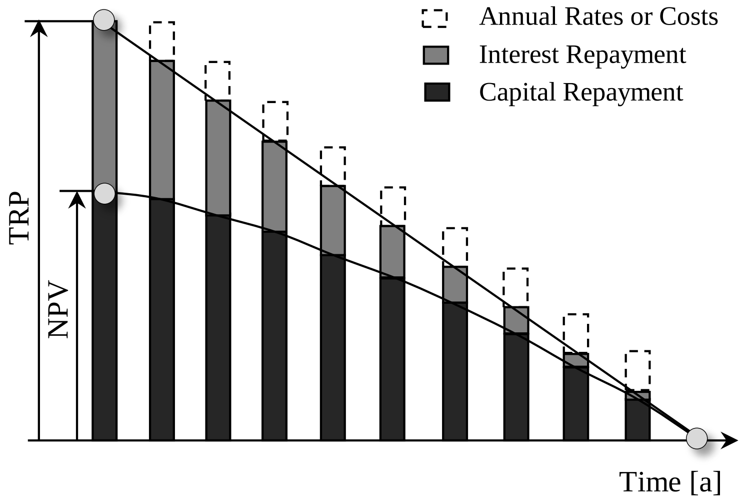

Figure 28: Calculation principle of annual costs during the standard life cycle of infrastructure assets [Kruschwitz, L. 2011]

$$R = NPV * (i * q^n) / (q^n - 1) \tag{25}$$

$$n = -\log[1 - (i * NPV / R)] / \log(1 + i) \tag{26}$$

$$TPR = R * n \tag{27}$$

When economic analyses are conducted during the life cycle of infrastructure assets, *years* or *even decades* are often between *single rehabilitation investments*, also involving changes in general market prices. Thus, for a long-term investment strategy, all significant price developments *of method elements* have to be taken into account. This is usually carried out *via price indices*, which reflect a *relative price development* in relation to a *reference year*. An accurate evaluation of any investment options therefore includes price adjustments. The calculation of the price index can be calculated based on equation (28) and is illustrated in Figure 29.

$$I_t = \frac{\sum_{j=1}^{n} p_{j,t} * q_{j,0}}{\sum_{j=1}^{n} p_{j,0} * q_{j,0}} \tag{28}$$

I_t … price index market basket in the reporting year
$p_{j,t}$ … price of ware j in the reporting year
$p_{j,0}$ … price of ware j in the reference year
$q_{j,0}$ … share of ware j in the reference year
i_t … average annual nominal growth
i_e … average annual effective growth

$$i_t = I_t^{\frac{1}{t}} - 1 \qquad i_e = (1 + \frac{i}{x_a}) - 1$$

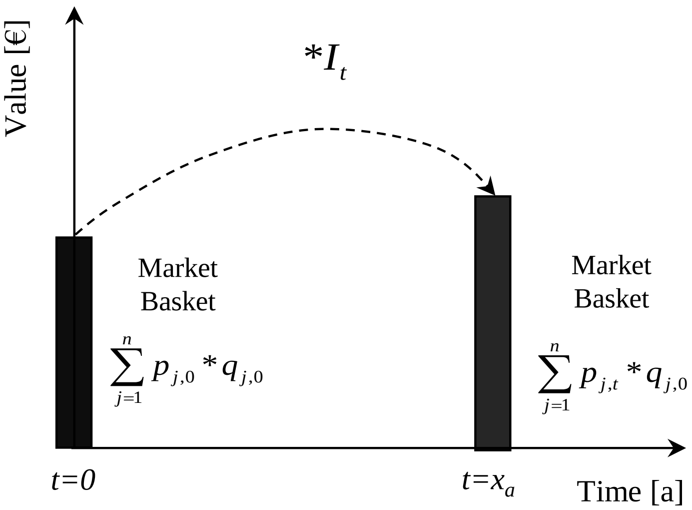

Figure 29: Price adjustment and index increase [Kruschwitz, L. 2011]

The *unit costs* of maintenance and rehabilitation measures, together with their expected *impact duration* are of crucial importance for an economic assessment in infrastructure asset management. Generally, unit costs of any measure show deviation regarding costs and extent (2.3.13). Aside from progressive and proportional cost development, *unit costs* C_r in practice tend to *decrease* with *growing extent* E_r due to a distribution of overhead costs described by *economy of scale*. Scale economy can be defined as the reduction in average cost per unit output for every increment in output [Labi, S. 2014]. Based on actual cost data, cost functions can be derived either by using common power-functions via equation (29) or specific types of cost functions, which are illustrated in Figure 26. *Power cost functions* can be used flexibly, where β_4 represents *fixed costs* and β_5 and β_6 are *regression parameters*. Marginal cost curves are equal to the differential of total cost curves and mark the costs for an additional unit. In the cost model illustrated here, marginal costs are considered as unit costs of an infinitely large measure extent and indicate the lowest possible costs.

$$C_r = \beta_4 + \beta_5 * E_r^{\beta_6} \tag{29}$$

C_r … unit costs of rehabilitation
E_r … measure extent
$\beta_{4;5;6}$ … regression factors based on actual rehabilitation costs
C_0 … overhead costs (~β_4)
C_m … marginal costs

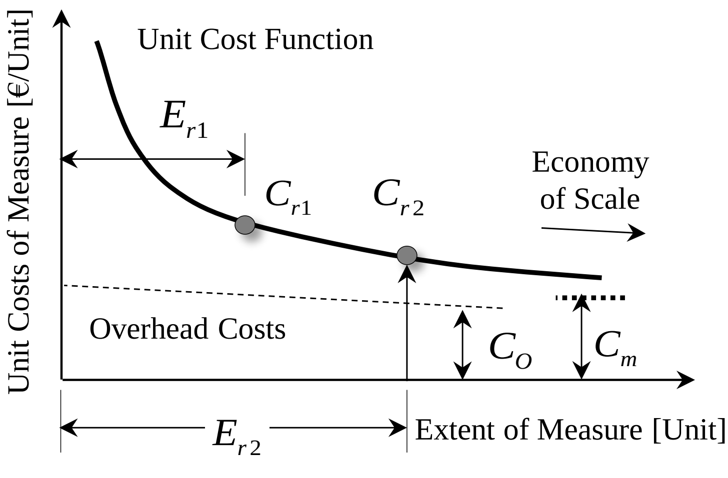

Figure 30: Deterministic model of construction or maintenance and rehabilitation unit costs based on measures extent [Kruschwitz, L. 2011]

For the comparison of investments based on dynamic methods of investment appraisal the duration of measure impact and the service life of the construction are of crucial importance. Thus, the determination of monetization timing of costs (= implementation time of rehabilitation or maintenance measures) and the determination of the expected service life is essential for a discounting of monetized impacts at decision time and thus for the net present value as well.

When decisions are made between *investment-alternatives*, which offer an *equal service life* and only include costs, the rehabilitation strategy with the *lowest positive* net present value should be preferred. However, if the benefits are also taken into account, the investment alternative with the highest positive net present value is considered advantageous.

For investment options, with a different service life (or durations of measure impact), priority should be given to the option with the *highest positive* annuity if, in addition to costs, revenues (benefits) are also included in the consideration. By contrast, if only costs of alternative measures are compared, priority should be given to the measure with the *lowest annuity*. Thus, the investment option for infrastructures, with the least annual costs can be found using equations (30) and (31), according to Figure 31.

In order to find the economic optimum, all possible measures and combinations of measures for all failure types with varying extent and timing have to be compared (compare chapter 2.3.16).

$$C_{a,c} = \frac{i*q^{x_{sl}}}{q^{x_{sl}}-1}*C_c\sum_{x=0}^{x_{sl}}(1+q^{-x_a}+q^{-2x_a}...) \tag{30}$$

$$C_{a,r} = \frac{i*q^{x_{sl}}}{q^{x_{sl}}-1}\sum_{x=0}^{x_{sl}}(C_c+C_r*q^{-x_c}+...) \tag{31}$$

i	… interest rate (2-4% recommended) with $q = 1 + i$	$C_{a,c}$	… annual costs perpetual reconstruction
x_{SL}	… service life with measures	$C_{a,r}$	… annual costs rehabilitation
C_c	… (re-) construction costs	C_r	… rehabilitation costs

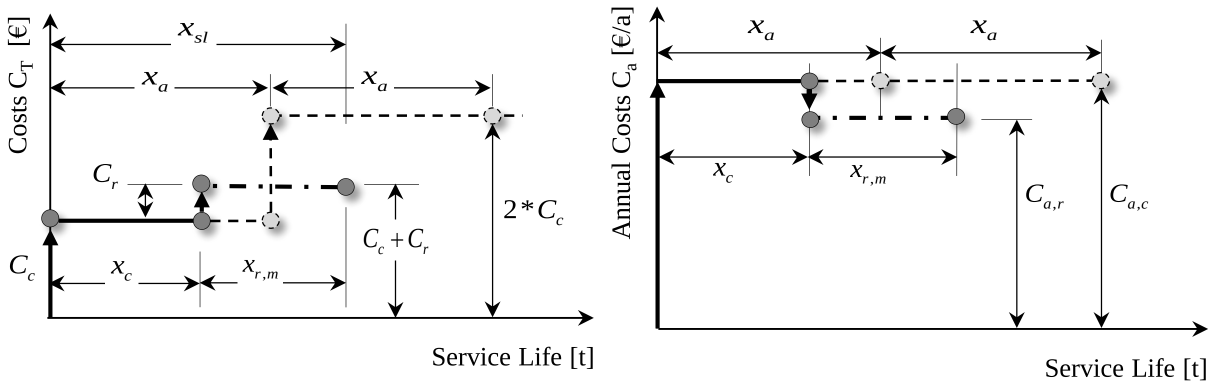

Figure 31: Simplified deterministic cost development and resulting annual cost for rehabilitation and renewal [Hoffmann M. 2013b]

2.3.15 Ranking, prioritization and budgeting

If the measure selection on infrastructure asset level is applied to the entire infrastructure route or network, a list of measures with a necessary budget in the selected time horizon can be obtained (measure program). This measure program is essentially concerned with the specification of the planned measures. Due to tight budgets, it is in practice often required to set priorities in the actual measure program. These priorities may be based on selection of projects that need to be done in order to avoid failures or safety risks, and those with the least annual costs compared to doing nothing. Depending on the time horizon of the measure program (one or more years) and the possibility to model combinations of construction, operation and maintenance measures as well as their timing, a distinction is made between *ranking* (redone each year) and *prioritization* processes (long-term effects are considered).

The *ranking* of these measures within the respective measure program may be determined by either one or more criterions, using, for instance, a scoring system for the evaluation. Possible ranking criteria could be asset condition (*serve worst first*), measure costs (*least expensive measures first*) or *favorable life-cycle costs*. After the ranking procedure, envisaged measures are successively executed until the budget of a period is completely depleted. This ranking process must be applied anew for each budget period. As an advantage of ranking methods, the easy comprehension may be mentioned. As a disadvantage, however, an unattainable uniform condition level may be named, as well as a suboptimal allocation of resources.

Prioritization generally refers to the *primacy* of an issue or a rating scale toward another. This may be interpreted in a temporal sense (*urgency*) as well as in a sense of *importance*. The urgency of measure implementation can be based on a subjective assessment or an objective background. Priorities are always established in line with the objectives of the infrastructure asset management approach. As a part of prioritization of investment variants, cost-benefit considerations may provide the basis for defining what is important and what is not. Within the prioritization process, scarce resources are used in an effective and cost-efficient manner over a period of several years, also taking into account long-term effects (such as impact duration). This is achieved by mathematical modeling of combinations of construction, operation and maintenance measures based on cost-benefit analysis, also including timing of measures. Within the comparison of investment variants as a part of prioritization, the ranking of implementation turn of measures is possible as well. As a result, maintenance options may be developed for different budget levels.

The complex programming process required for this purpose may be considered as a disadvantage of prioritization methods. An optimization of a network towards a uniform condition level is also not possible by means of prioritization methods. Furthermore, prioritization methods do not provide the opportunity to adapt the overall investment strategy in asset management in terms of the actual available budget.

If a program of measures, aiming at an increased level of asset condition, cannot be financed over several years, procedures to provide additional budgetary resources have to be developed. Therefore it is necessary to model the condition development with and without additional financial resources. The basis for a sound argumentation of investment decisions may be derived based on different optimization options presented in chapter 2.3.16.

2.3.16 Optimization in infrastructure asset management

The term *optimum* in infrastructure asset management is defined as the *best achievable result* in the sense of *a compromise* between different target parameters or criteria, either from the point of application, or an asset management objective. The identification of the optimum, under given constraints and objectives, is defined as optimization[21]. In mathematics, *optimization* (alternative mathematical programming) is the selection of a *best solution* (with regard to some criteria) from some set of available alternatives[22]. Figure 32 shows the basic concept of optimization in infrastructure management, where a set of decision variables is used to realize the respective objective. In practice, decisions are often made facing certain boundaries or constraints (financial, physical, institutional and political). Thus, in a typical problem structure in infrastructure asset management, both the *objective* (e.g. cost functions) and the *constraints* (e.g. fixed budget) are expressed as a function of the *decision variables* (e.g. performance parameters). The attribute of the system that is to be optimized may either be a *physical component* (e.g. cross section of a road or channel) or an *abstract component* (e.g. system speed or asset costs) [Labi, S. 2014]. As an essential advantage compared to simple ranking and prioritization, optimization methods allow an adaption of the investment strategy with respect to budget restrictions.

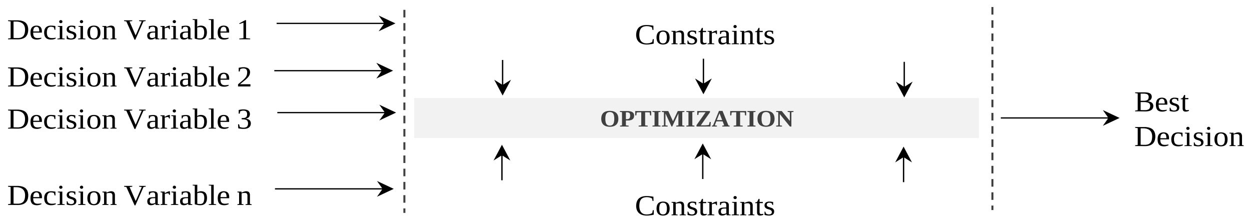

Figure 32: In the basic concept of optimization values of the objective functions (s) yielded by each combination of the decision variables are compared. The best decision is a statement of the best combination of values of the decision variables and the corresponding value of the objective function(s) [Labi, S. 2014].

From a mathematical point of view, optimization means consequently that an *objective function* is *minimized* or *maximized* by varying the respective input parameters. For a *two-dimensional* optimization problem (with two independent input parameters) the objective function is described by a surface in the three-dimensional coordinate space, where the varying input parameters represent the length and depth axis. The height subsequently represents the minimum or maximum value of the objective function that has to be determined. The *effort* to solve optimization problems and identify global *minimum* and *maximum values* crucially depends on the *shape of a surface*. The simplest case of a two dimensional optimization problem is described by a paraboloid with one global maximum

$$z = f(x, y) = -(x^2 + y^2) + 4$$

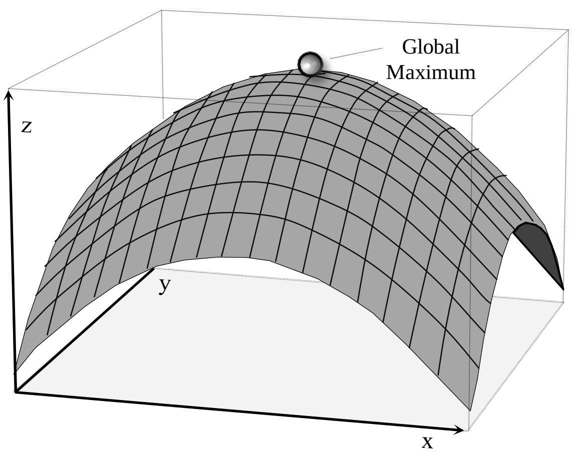

Figure 33: Global maximum (=local optimum) of a convex optimization problem for a 3D surface of a paraboloid

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[21] https://de.wikipedia.org/wiki/Optimum; (Last access: 10.06.2015)
[22] https://en.wikipedia.org/wiki/Mathematical_optimization; (Last access: 10.06.2015)

value (Figure 33).

Depending on the respective *topology*, *several local minima* or *maxima* may exist for a surface. Finding the next relative minimum or maximum in the neighborhood is referred to as local optimization. A local minimum is defined as a point for which there exists some $\delta > 0$ for all x such that

$$\left\| x - x^{*} \right\| \leq \delta \qquad \text{and} \qquad \mathrm{f}(\mathrm{x}^{*}) \leq \mathrm{f}(\mathrm{x}) \tag{32}$$

This is true on some region around x^*, where all of the function values are greater than or equal to the value at that point[23]. The two tasks show a different level of difficulty. For local optimization problems, a number of methods are available, which may be implemented very fast. For global optimization problems, the *solvability* heavily depends on the topology of the objective function. For the specific task of *convex optimization*, any local optimum is equal to the global optimum. In a Euclidian space, a region is defined as convex if, for every pair of points within the region, every point on a straight line segment joining the two points is also within the region [Labi, S. 2014].

From an economical point of view, *rational principle*, describing the *ratio input* to *output* may be applied in infrastructure asset management in three forms, also referred to as *minimum principle*, *maximum principle* and *optimum principle*[24].

According to the *minimum principle*, the result (=output, e.g. given target condition) is predetermined. Consequently, the allocation of resources (=input, e.g. financial resources) should be minimized in order to achieve the target output (e.g. target condition). In waterway asset management, the minimum principle corresponds to existing agreements and recommendations for fairway parameters, such as the European Agreement on Main Inland Waterways (AGN), can be considered as desired solution from the point of view of customers, which strive for reliable defined target fairway parameters.

The maximum principle requires that the allocation of resources (=input; e.g. given budget) is predetermined. Therefore, the goal is to generate a maximized result (=output; best possible condition). In fact, the maximum principle therefore corresponds to the lived reality of the most Danube waterway authorities, which are trying to provide the best possible fairway conditions for inland navigation, despite their limited budgets.

The *optimum principle* represents a combination of minimum and maximum principles wherein both input and out are *variable*. This principle aims at an *optimized relationship* between *allocated resources* (e.g. low maintenance cost) and *target benefits* (e.g. low cost for users). In practice, complex optimization problems in assessing the cost-effectiveness of infrastructure investments subsequently require the application of the optimum principle. Applying the optimum principle for inland waterways would mean that both measure costs and transport costs have to be varied, and optimal fairway parameters are subsequently derived by minimizing total cost.

In the case of infrastructure asset management, the optimization problem, with a necessary *trade-off* between *maintenance costs* and *user costs*, can be solved by the identification of the minimum of the *one-dimensional* analytical function *f(x)* of total costs, based on the determination of the *zeros of the first derivate* of the total cost function (Figure 34) using equations (60) to (62).

[23] *https://en.wikipedia.org/wiki/Mathematical_optimization; (Last access:10.06.2015)*
[24] *https://de.wikipedia.org/wiki/%C3%96konomisches_Prinzip; (Last access:10.06.2015)*

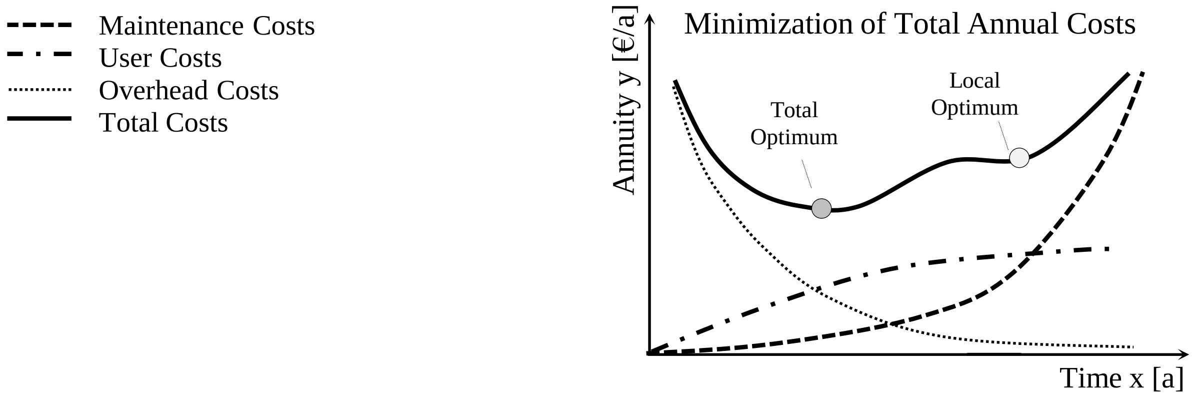

Figure 34: Optimization of budget allocation by determination of the mathematical minimum of the objective function of total costs; adapted from [Labi, S. 2014].

For a general *unconstrained optimization problem*, where $f(x_1,x_2,...x_n)$ is a *nonlinear* function of *n* decision variables, $x_1, x_2, \ldots, x_n$ the minimum or maximum value of the objective function must accomplish the following conditions:

$$y = f(x) = \min! \qquad y = f(x) = \max! \tag{33}$$

$$y' = f'(x) = 0 \qquad y' = f'(x) = 0 \tag{34}$$

$$y'' = f''(x) > 0 \qquad y'' = f''(x) < 0 \tag{35}$$

If the objective function shows a zero slope then the *optimum-decision* variable is found. Depending on the structure of the objective function and constraints as well as the nature of the decision variable, optimization problems may further be divided into *linear problems* (both the objective function and all constraints are linear expressions of the decision variable) and *nonlinear problems* (the objective function and / or at least one constraint are nonlinear) [Labi S. 2014].

For optimization problems with *constraints*, substitution techniques are useful for an analytical solution of nonlinear optimization problems having equality constraints. With an objective function *y=f(x),* constraint functions, $g_i(x)$ and a decision variable *x*, the optimization problem can be illustrated as follows:

$$y = f(x_1, x_2, \ldots, x_n) \tag{36}$$

$$\left.\begin{array}{l} g_1 = f(x_1, x_2, \ldots, x_n) = b_1 \\ g_2 = f(x_1, x_2, \ldots, x_n) = b_2 \\ g_m = f(x_1, x_2, \ldots, x_n) = b_m \end{array}\right\} \tag{37}$$

The classical optimization problem may be solved for *m (m<n)* variables in terms of the remaining *(n-m)* variables.

Optimization problems may further involve only one objective, also called *single-objective optimization* (the objective function involves a specific performance attribute such as average network condition) or several, often conflicting objectives, also referred to as *multi-objective optimization* (the objective function involves also one specific performance attribute that is optimized towards different objectives; e. g. maximization of minimum network condition and minimization of the range of asset conditions) [Labi, S. 2014].

Furthermore, the structure of an optimization problem becomes more demanding with an increasing number of performance attributes. For *single-attribute optimization* problems, only one performance attribute, as for example average condition performance is subject of the optimization. *Multi-attribute optimization* by contrast involves a multiplicity of considerations that result from owner policy, stakeholder concerns and infrastructure type [Labi, S. 2014]. The research field of multiple-attribute decision making is concerned with mathematical optimization problems involving *more than one* objective function to be optimized *simultaneously*.

By invoking inland navigation, for example, typical attributes that are to be optimized may involve, for instance *fairway availability*, *necessary maintenance expenditures* and *environmental impacts*. In the past, the cost (initial or life cycle) has largely served as a dominant attribute in decision making. However, recognizing, that the cost attribute alone may not provide robust, sustainable, and universally acceptable decisions, resulted in the fact that multi-attribute optimization is increasingly gaining importance in infrastructure asset management [Prickell, S. et al. 2000].

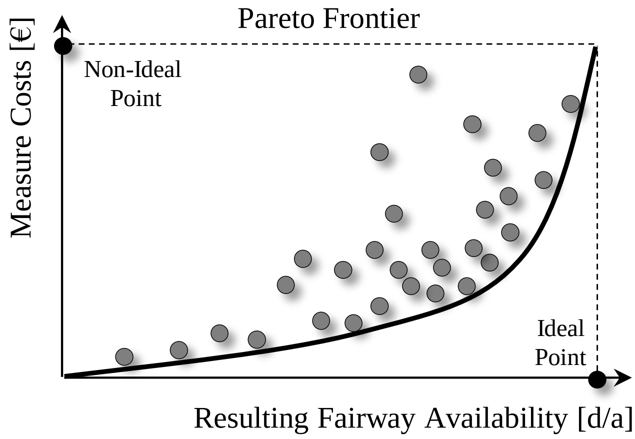

Figure 35: Visualization of the Pareto-frontier for fairway availability which only may be increased if additional measures are implemented and the effort for waterway authorities is degraded.

For some *nontrivial multi-criteria optimizations problems*, however, one single solution does not exist that simultaneously optimizes each objective. In that case, the objective functions are considered as conflicting, and an infinite number of *Pareto-optimal* solutions exists. The quantity of *Pareto-optimal* points is termed as *Pareto-Frontier*. A solution is called Pareto-optimal if none of the objective functions can be improved in value without degrading some of the other objective values. For operators of waterways this would mean, for example, that for given fairway parameters, an increase in fairway availability is not possible simultaneously with a reduction of measure costs. Without additional preference information, all Pareto-optimal solutions are considered equally good.

Multi-criteria optimization is applied in infrastructure asset management when optimal decisions have to be conducted under consideration of *trade-offs* between two or more conflicting objectives (e.g. maximizing infrastructure performance whilst minimizing maintenance cost or maximizing revenues and minimizing failure risk).The introduction of *additional criteria* may facilitate the decision-making process in many cases. In order to solve a multi-criteria problem, common methods attempt to convert the original problem with multiple attributes into a single-attribute problem by *scalarization.* Scalarizing a multi-attribute optimization problem means formulating a single-objective problem such that optimal solutions to the single-attribute optimization problem are Pareto-optimal solutions to the multi-attribute optimization problem. The most preferred solution may be found using different philosophies, including so-called *no preference solutions* requiring a decision maker, *a priori*, *a posteriori* and *interactive* methods[25].

For inland waterways, the concept may be applied for the case of a given budget where a number of different combinations of fairway width and depth may be reached on entire river stretches by the implementation of appropriate dredging measures within the same budget. A greater depth means that less budget is available for dredging a greater fairway width. For operators of waterways, these solutions are equal in terms of budget. For navigation companies, however, each solution results in certain transport costs, meaning the optimal combination of fairway parameters within the budget offers the lowest transport costs.

For complex problems, an analytical solution of optimization problems is often not feasible, in the sense of a comprehensive examination of all influencing variables, meaning that computational optimization techniques have to be applied. The research field of applied mathematics and numerical analysis already developed algorithms for optimization that may be applied in infrastructure asset management. To solve optimization problems, researchers may use either algorithms that terminate in a *finite number of steps* (e.g. simplex), *iterative methods* that converge to a solution (e.g. Newton's method or interior point methods), or *heuristics* (e.g. memetic algorithm or differential evolution) that may provide approximate solutions to some problems.

Where the relationships between the objective function and the decision variables are not so clear or not explicitly stated in mathematical terms, optimization by repeated simulation, which is described as a laborious process, is considered as most useful. Thus, optimization can involve an extensive search before arriving at the optimal solution by quantitatively examining each and every possible scenario, establishing the output of the objective function that corresponds to each scenario, and identifying the best solution.

Moreover, in infrastructure asset management, all infrastructure operators have to deal with the so-called *knapsack optimization problem*. The knapsack problem KP describes constrained optimization problems (e.g. given budget) where, for a set of items (e.g. measures) that each have a weight (e.g. costs) and a user value (e.g. user benefits), a subset (e.g. certain measures) are selected whose total weight (e.g. total measure costs) does not exceed a given weight constraint (e.g. given budget). Under the condition of one given constraint (single size constraint=simple knapsack), such as a given budget, the benefits (user value) through the selection of measures should be maximized, meaning the most favorable measure should be realized within the given budget.

When, for each measure, two or more alternative measures are available, the problem is termed as *multi-choice knapsack* (MCKP). The problem is referred to as *multi-dimensional knapsack* problem (MDKP), if there is more than one constraint such as maximal budget and performance targets.

For *multi-choice multi-dimensional knapsack* problems (MCMDKP) decision makers are not only faced with multiple choices for each alternative, but also have to contend with constraints in at least two dimensions. For infrastructure systems, this knapsack problem is considered as most common and realistic [Labi, S. 2014]. The same holds true for inland waterways.

A MCMDKP is given due to a constrained given budget and the question, which measure should be implemented where and to which amount so that the availability performance is maximized for users. Due to the fact that inland waterways constitute a serial system where the shallowest section limits the utilization of

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[25] *https://en.wikipedia.org/wiki/Multi-objective_optimization; (Last access:10.06.2015)*

the entire fleet, the budget has to be spent aiming at a uniform fairway depth on the entire river stretch. Which fairway width may be achieved depends on the level of budget. Solving this knapsack problem consequently means that starting with a fairway channel with a defined depth and minimum fairway width (deep fairway channel), the fairway width may be expanded stepwise until the budget is spent and thereby determining all necessary measures together with their extent.

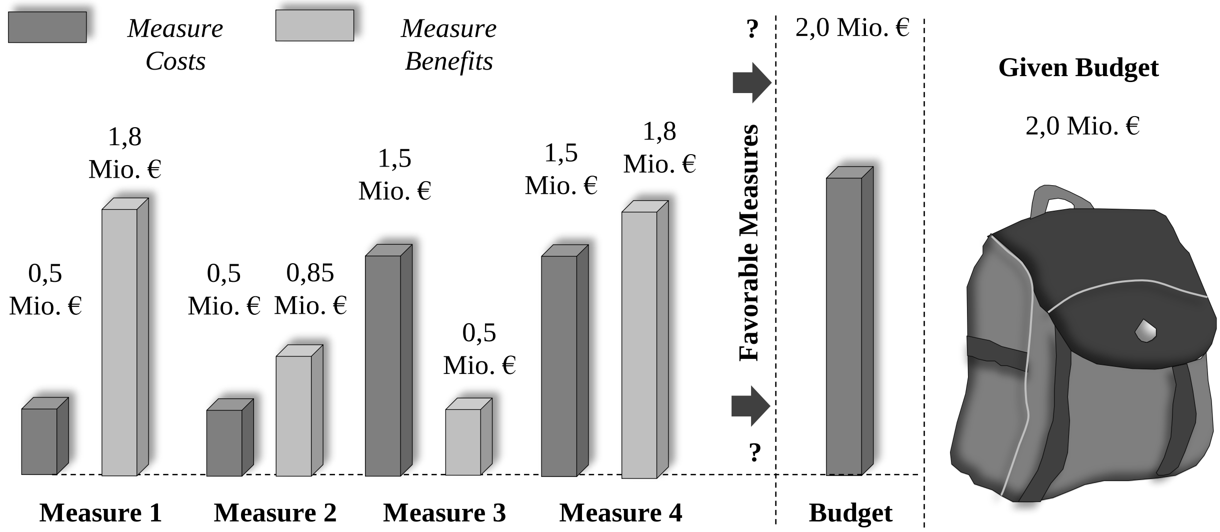

Figure 36: Illustration of a simple knapsack problem with a given budget constraint and a number of measures that must fulfill the budget constraint while the benefit of measures must amount to a maximum value.

The presented waterway asset management approach accounts for all essential optimization problems and allows both the selection of the optimal measure for a specific shallow sections as well as the determination of the optimized measure program for the entire Danube, and also incorporates the possibility to optimize in terms of the minimum principle, the maximum principle and the optimum principle.

The optimal combination of measures and timing for single shallow sections is based on minimal annual costs. For the optimization algorithm a stepwise calculation of condition performance, with and without measures, as well as resulting annual costs is necessary. The optimal timing within the selected time horizon is found based on the least annual cost without failure. The given interest rate is often decisive for an economic application of preventive measures or additional investments to extend service life [Hoffmann, M. 2006].

Apart from timing of measure implementation, the optimization of maintenance length or work-zone length is of a high practical importance for any infrastructure. Contemporary approaches for optimized work-zone length use cost functions for all applicable measures. Starting with the smallest section length from a condition survey, the measures with the least annual cost are selected. In the further steps, the section length is extended and the measure combination with least annual cost is calculated again. If the annual costs of the extended infrastructure section are lower than those of the individual single infrastructure sections, then the section length is extended further, until a different measure, or timing on singles infrastructure sections, yields lower annual costs.

2.4 Overview – State-of-the-art in waterway asset management approaches

Waterway asset management is a relatively new discipline in infrastructure asset management, but, until recently, a uniform and consistent approach there was missing for managing inland waterways due to the variety of *organization types* of waterway agencies, *main tasks* and *responsibilities*, *river types and regimes*, *river engineering structures* and *waterway assets* as locks and weirs. In chapter 2.3.3, the general structure of asset management systems, as well as required modules for the implementation of such systems, were already discussed.

The following sections will provide a summary of the state of development of waterway management systems as an anchoring point for the introduction of the asset management approach for inland waterways, which was developed for the Austrian waterway administration VIADONAU starting in autumn 2012. For this purpose, a number of implementation examples are discussed in terms of already available modules and missing processes, resources and data. As an example, particularly the Danube waterway administrations will be cited, as these already have been examined in the context of the project "NEWADA duo", co-funded by the European Union, regarding existing lacks for the implementation of a Danube-wide waterway asset management system.

Numerous examples of waterway operators provide evidence, that although river engineering structures and locks offer a very long service life; the pressure to establish appropriate management systems has increased over the recent years due to increasing asset age. Usually, the design and implementation of an assets inventory systems, including all relevant assets, turns out as the first project towards building up a comprehensive management system. At this stage, many operators already encounter the problem that asset age and partly also construction details are unknown. Depending on the respective business goals and common inspection guidelines, condition data and asset information are subsequently collected in a second step. Due to the complexity of waterway infrastructure systems and a high number of conflicting tasks, it is often not clear what level of detail and which data format is necessary for further evaluations. Consequently, most systems suffer from missing concepts of future data assessment, evaluation and analysis.

Furthermore, the *World Association for Waterborne Transport Infrastructure* (PIANC) developed a number of reports, containing a compilation of the most essential foundations for waterway asset management approaches. However, these reports are mainly concerned with infrastructure assets associated with waterways such as *locks*, *bridges*, *weirs* and river engineering structures, but not with the *navigability of waterways* itself. The covered aspects are rather general and include the structure of asset inventory systems, required data, inspection frequencies, methods for condition assessment and grading, condition prediction methods and assessment criteria for the derivation of intervention thresholds as well as typical maintenance intervals. According to a survey of PIANC, only 10 percent of participating organizations consider the bed of their waterway as essential for their waterway asset management system [PIANC 2013].

The navigability of the fairway channel is an important *core issue* in waterway management, especially for *near-natural rivers*. Globally, there are only a few near-natural rivers which are of considerable importance for the shipping industry. Consequently, there were no approaches available which strived for an increase or optimization of the performance of the fairway as the core of inland waterway transport.

2.4.1 State-of-the-art in waterway asset management Danube

For all *near-natural rivers* with considerable importance for the shipping industry, maintaining the availability of the fairway channel for inland navigation must be the methodological core of any waterway asset management system. *Availabilit*y at the center of a waterway asset management approach implies that a fairway with predefined widths and depths can be used for navigation without any further restrictions. Currently, there are *no known systematic waterway asset management* approaches available, which allow an optimization regarding fairway availability and performance.

According to waterway authorities [VIADONAU 2013b], Danube navigation is only shut down in times of extreme weather conditions (floods, extensive ice formation) and only in a few Danube countries at low-water conditions. Subsequently, the current definition of availability of the fairway is a *yes/no criteria* in days per year. Based on this definition, the average availability of the Austrian section of the Danube waterway for the time period from 1999 to 2013 was calculated with 97.8% [VIADONAU 2014].

However, the *explanatory value* of this existing availability concept is rather limited in terms of fairway performance due to the fact that *insufficient fairway depths* in *low-water periods* are not considered. Furthermore, a yes/no criterion does not allow an assessment of marginal to substantial improvements. On the other hand there are a number of international recommendations and agreements (e.g. UNECE 1996, Danube Commission 1988, Danube Commission 2013) aiming at common minimum fairway depths and widths on a certain number of days per year, setting ambitious targets for fairway maintenance.

Due to political, technical, environmental and economic reasons, this recommended availability performance is *almost never met in practice on all river stretches* of the riparian countries. Unfortunately, exact calculations of available fairway depths being closely related to possible draughts loaded of individual vessels and convoys do not exist currently. Furthermore, there are no systematic approaches available allowing an assessment of the actual availability of fairway parameters. Thus, maintenance and river engineering works are mainly planned on *a case-by-case basis* with limited continuous coordination and optimization between riparian countries. Additionally there is a fundamental lack of basic data models and software solutions that would enable an assessment of measure impact on fairway availability and, thus, an optimization of all kinds of measures [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

For passenger transport in the tourist season from the beginning of April to the end of October with a vessel draught usually not exceeding 1.5 m, the current yes/no availability criteria might be sufficient even at low-water conditions. In comparison, the possible utilization in goods transport, and the resulting draught, heavily depends on actual available fairway depths. If possible, utilization due to insufficient fairway conditions falls beyond *a certain margin* inland navigation is no longer competitive compared to other modes of transport. With typical transport distances between 500 to 2,000 km and transport times of one to three weeks, a few days per year with insufficient fairway depths might be economically manageable for the navigation industry. However, if the resulting delays are not manageable anymore, and the average load factor of the vessel fleet during the year falls below 50 to 60%, then navigation on the Danube might not be able to stay in the market in the long run. Therefore, any kind of approach that only uses yes/no availability criteria is not capable of an assessment and optimization of measures regarding the needs of the transport industry and resulting transport costs [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]

2.4.2 Existing approaches in fairway maintenance management –Danube River

As a first step towards a systematic maintenance management for their fairway, the Danube waterway agencies all implemented a cyclic maintenance approach based *on best practice knowledge*. Any kind of maintenance management, as part of a *life-cycle-cost-based* asset management of infrastructures, is defined by a number of goals and certain steps or modules in a circular process leading to a *constant improvement* based on an analysis of previous experience and results. For fairway maintenance, the basic process consists of the monitoring and surveying of fairway conditions, assessment of current conditions and an estimation of possible developments as basis *for planning and optimization of necessary maintenance or engineering measures.* Depending on priorities, these measures have to be executed within given budgets followed by an assessment of results compared to predefined performance goals. A continuous improvement in this *circular process* is achieved on the basis of an extensive documentation followed by a persistent analysis and implementation of recognized improvement potentials. Management in this context means providing *leadership and directions* in order to keep this cycle running while gaining the means for evidence-based management decisions (Figure 37).

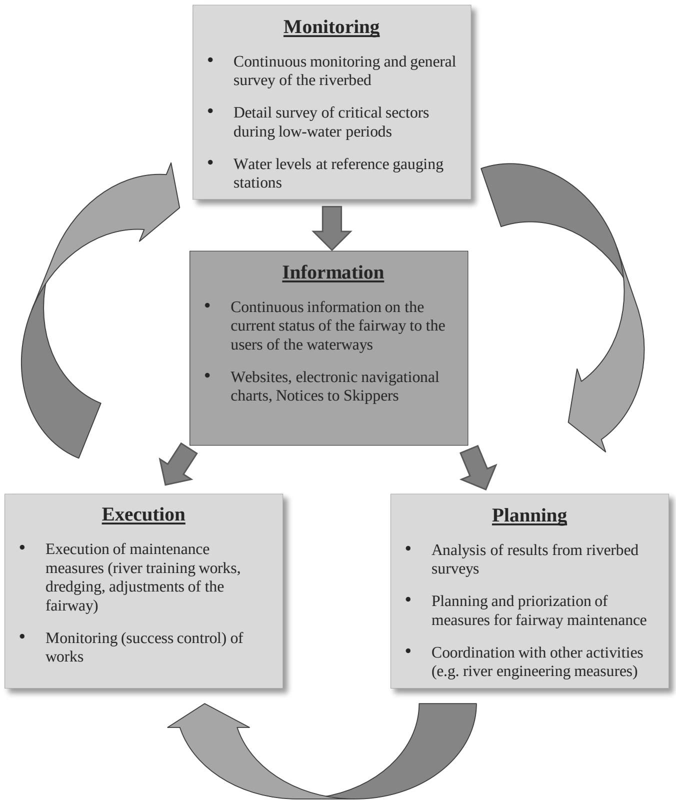

Figure 37: Overview of common empiric fairway maintenance cycle; adapted from [VIADONAU, 2013 b]

The general goal of fairway maintenance should be providing *optimal continuous conditions* for inland navigation – especially in low-water periods – based on an *effective use of available resources*. In practice, waterway authorities are operating under several international agreements and recommendations regarding targeted fairway widths and depths on a certain number of days per year. Whether these targets may be achieved or not depends on a number of factors that are, to a certain extent, *beyond the range of waterway authorities* (e.g. water levels during the year, available budgets etc.). Additionally, there is no common approach available that allows for an assessment of the efficiency of measures and any target conditions. For waterway agencies with sufficient budgets, this leads to the situation that an assessment of possible alternatives and optimization to achieve these goals is rather difficult. If the budgets are insufficient, then recommended goals cannot be achieved at all, leading to the question how and to what end these insufficient resources should be used. Without any positive or negative consequences for gaps between targeted and achieved results, a systematic improvement is not likely, leading to an empirical trial-and-error approach with given rules.

Any infrastructure maintenance cycle always starts with an inventory and survey of the current conditions. The *optimal frequency* of any kind of survey is found if the additional costs for surveys are not outweighed by the *benefits of better decisions* based on these additional surveys.

Starting with the survey of fairway conditions the available equipment and assessment performance shows large deviations in accuracy and period length between assessments ranging from every two months, in critical sections with multi-beam up, to an assessment every two years, with *single-beam* and *echo sounder*. The subsequent processing capacity leading to navigation charts or action plans depends on the length of a river section and may range from one day to several weeks. Cleary, the existing practical survey approaches are therefore not the result of an optimization process but in fact the result of both, *empirical experience* and *available resources*.

For any kind of decision process regarding measure implementation, an assessment and estimation of possible condition development, with and without measures, is crucial. A comparison and optimization of all technically feasible measures as a result is therefore only possible if both, *costs and impacts (duration)* are known. If the goal of measures is to improve fairway availability, then the question must be: Which improvement might be attainable with which type, extent and costs of measures and for what time frame? Without a systematic database including implemented measures and enabling a mathematical description, both impacts and costs of measures are subject to individual empirical experience. Currently, there is some information available in waterway agencies regarding the costs and time of measure implementation. As to the duration of impact and condition development there are only selective project-based assessments but no systematic condition-prediction approaches available. Therefore, a systematic optimization of operation, maintenance and engineering works is currently not feasible, leading to an empirical and budget-driven priority approach.

In contrast to an optimization process, setting priorities for measures on transport infrastructures basically means a ranking (i.e. regarding the highest negative impact on infrastructure users) the worst condition compared to a target level or the highest losses due to malfunction. Typical priorities regarding fairway maintenance are giving to measures on shallow sections with the lowest fairway depth compared to *low navigable water level* (LNWL). Additional criteria may be the *remaining fairway width with sufficient depth* and/or the *rate of sedimentation* on critical bottlenecks based on an estimation of remaining time until the section cannot be passed. Currently, these processes are handled manually case-by-case with the use of different software tools. Due to the *massive amount of necessary data and necessary logistic efforts*, *setting the priorities* in waterway maintenance is mainly the result of *experts discussing* the implementation of planned measures.

In general, management and implementation of fairway maintenance on a dynamic river on a few hundred kilometers with constantly changing riverbed morphology and water levels is, in itself, a very demanding task. Only with *sufficient equipment, trained staff and a comprehensive holistic approach* will this task become manageable in a modern sense of a waterway asset management.

Nonetheless, this is not enough to provide continuous fairway conditions, if fairway maintenance on a comparable and coordinated level is not performed in all riparian countries alike. For shippers and navigation companies, on the other hand, the maintenance approach does not matter as long *as continuous fairway conditions* and *actual reliable and accessible information* are provided. If this information would be available, navigation companies might calculate *with lower safety margins* leading to a higher load factor of vessels throughout the year. Due to the amount of fixed costs, this would lead to *considerably lower transport unit costs* and *higher competitiveness* on the transport market [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

2.4.3 Setbacks of current approaches in waterway asset management –Danube River

The following chapter of this thesis will provide a SWOT-analysis of existing waterway asset management approaches of Danube waterway agencies regarding main modules of such approaches. As a strength regarding the topic basic data, it should be mentioned that generally all important current and historical data on riverbed, water level and fairway are digitally available. In addition the basic equipment for monitoring of fairway conditions is available at all waterway authorities and critical sections are surveyed with an increasing frequency. Further potential improvements can be expected from the *establishment* of a *common database for riverbed surveys and water level data* which is currently at the beginning. Different *projection systems* and the use of *different reference altitudes* in the riparian Danube countries until now still represent a major challenge. Common standards for marking plans and electronic navigation charts have already been implemented by all Danube waterway agencies. Unfortunately, this does not yet apply for further data processing and storage of riverbed surveys. As a weakness, it must be mentioned that multi-beam surveys for more accurate analyses are currently available only in some Danube countries.

A SWOT analysis in the thematic field of availability and bottlenecks shows that there is unfortunately *no* current *possibility* for a comprehensive calculation of the actual availability of fairway parameters, as most evaluations are based on low navigable water level (LNWL). A further weakness of the Danube waterway management system is that a *sufficient monitoring frequency* for *critical sections* is not established on all river stretches. For this purpose, a unified methodology for monitoring and assessment of critical riverbed developments is missing as well. This also applies to a *consistent investment strategy*, which takes the systematic assessment of measure impact on fairway availability into account [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Table 1: Data management, survey capacity and processing capability in Danube waterway asset management [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]

STRENGTHS	WEAKNESSES
Basic data: ▪ Actual & historic data on riverbed surveys and water levels available ▪ Digital data on fairway path and location of signals etc.	**Basic data:** ▪ Very few common data standards, different projections ▪ Different reference altitudes, most single files / no database
Survey capability: ▪ Principal survey capacity for single beam & echo sounder ▪ periodic survey of riverbed & shorter interval for bottlenecks	**Survey capability:** ▪ Multi-beam surveys not in all agencies available ▪ Intensity and quality of surveys are largely different
Processing capability: ▪ Standard marking plans, plans of critical sections ▪ Data & information for ENCs (electronic navigational charts)	**Processing capability:** ▪ Standard plans with different processing time ▪ No common standards, capacity for specific analyses limited
OPPORTUNITIES	**THREATS**
Basic data: ▪ Develop harmonized standards and a common database for riverbed surveys and water level gauging data	**Basic data:** ▪ Continue on different levels without standards making availability & measure optimization impossible
Survey capability: ▪ Increase utilization of surveying capability ▪ Multi-beam surveying capability for all waterway agencies	**Survey capability:** ▪ Current survey intervals not often enough for actual assessment of fairway conditions and measure impacts
Processing capability: ▪ Training of staff and provision with adequate software ▪ Decrease processing time < one week = actual information	**Processing capability:** ▪ Slow processing time and different levels of quality lead to inefficient planning and unsatisfied customers / shippers

Table 2:SWOT performance indicators in Danube waterway asset management [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]

STRENGTHS	WEAKNESSES
Availability: ▪ Regular assessment of fairway parameters ▪ Publication of surveying results	**Availability:** ▪ Calculation of availability of fairway parameters currently not possible--everything related to LNWL
Bottlenecks: ▪ Bottlenecks are well known ▪ More frequent assessment of critical sectors common	**Bottlenecks:** ▪ Survey intensity shows large deviations ▪ No systematic monitoring for critical developments
Measures: ▪ Effective maintenance and engineering measures known ▪ Implementation of necessary measures in some countries	**Measures:** ▪ No consistent investment strategy--limited measure budget ▪ No systematic assessment of measure impact on availability
OPPORTUNITIES	**THREATS**
Availability: ▪ Providing continuous level of availability on whole Danube waterway ▪ Calculating current availability for entire transport routes	**Availability:** ▪ No improvement in actual availability ▪ No improvement in availability information for customers
Bottlenecks: ▪ Common standards for assessing critical sectors ▪ Common database on survey of bottlenecks	**Bottlenecks:** ▪ No improvement in processing capability ▪ Remaining critical sections prevent continuous conditions
Measures: ▪ Common strategy and investment in most critical sectors ▪ Necessary funds for measures	**Measures:** ▪ No common strategy and implementation of measures ▪ Continuous lack in sufficient funding for necessary engineering and maintenance measures

Table 3: SWOT measure implementation & controlling [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]

STRENGTHS	WEAKNESSES
Budgets: ▪ Budget for existing staff available	**Budgets:** ▪ Budget for necessary surveys and measures in almost all agencies not available
Implementation: ▪ Implementation of riverbed surveys ▪ Implementation of measures as well on the provision that budget is available	**Implementation:** ▪ Currently limited coordination of maintenance strategies ▪ Necessary measures not implemented--no budget
Result verification: ▪ Single- & multi-beam surveying prior and after measures established in some agencies	**Result verification:** ▪ Survey of measure impacts leaves improvement potential
OPPORTUNITIES	**THREATS**
Budgets: ▪ Increasing political awareness leads to more budget for necessary measures	**Budgets:** ▪ Budgets stay the same or are cut even further
Implementation: ▪ Coordinated priorities and efficient implementation of measures	**Implementation:** ▪ No implementation of necessary measures leads to low availability and declining inland waterway transport
Result verification: ▪ Systematic survey of measure impact and exchange of experience	**Result verification:** ▪ No systematic assessment of measure success due to a lack of funding, staff, training & equipment

3 INLAND WATERWAY ASSET MANAGEMENT

3.1 Overview waterway asset management

Waterway asset management is a *holistic multidisciplinary* approach for the *development*, *maintenance*, *rehabilitation and replacement* of *waterway assets* based on *life cycle costs*. Whether or not a waterway management approach qualifies as asset management therefore depends on the covered aspects. Figure 38 provides an overview of typical asset management organization structures and implementation cycle of tasks. The *strategic level* is the highest level of decision making where the general strategy and goals are set, the budgets and investment constraints are handled and the main projects are defined. On this level, there is a need for actual generalized figures and benchmarks as well as actual data from the main projects and their financial needs to enable the necessary steering and controlling.

The *management level* is responsible for certain tasks like hydrographical surveys, environmental protection or maintenance measures etc., or assets like river engineering structures, ports, flood protection dams. This medium level of decision making needs more detailed information, and has to implement a process-oriented asset management cycle from condition survey, prognosis, measure planning and optimization up to implementation and follow up of achieved results. The *implementation level* needs clear directions, based on checklists and protocols, in order to execute necessary tasks according to the overall strategy. Due to the fact that this level is the closest to the actual situation, the experience, motivation and feedback of the staff involved is crucial for any asset management.

Waterway asset management is not an end in itself, but may be seen as a service for the society in general, respectively the navigation sector, shippers, the economy, environment and the public alike. Thus, it is a difficult task to coordinate the different objectives of these stakeholders and align them into one unified optimized asset management strategy. Furthermore, with limited budgets there will always be a tradeoff regarding desirable and actually affordable conditions. The acceptance of both waterway asset management approaches and achieved results depends on an appropriate communication and participation of relevant stakeholders in shaping these asset management processes. In the following sections of this report, such an asset management approach for inland waterways, with the main emphasis on operation and maintenance of the fairway, is presented. The presented approach is availability and performance-based, and allows an optimization of any kind of necessary measures for different strategies and goals with a comprehensive life cycle costing approach [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

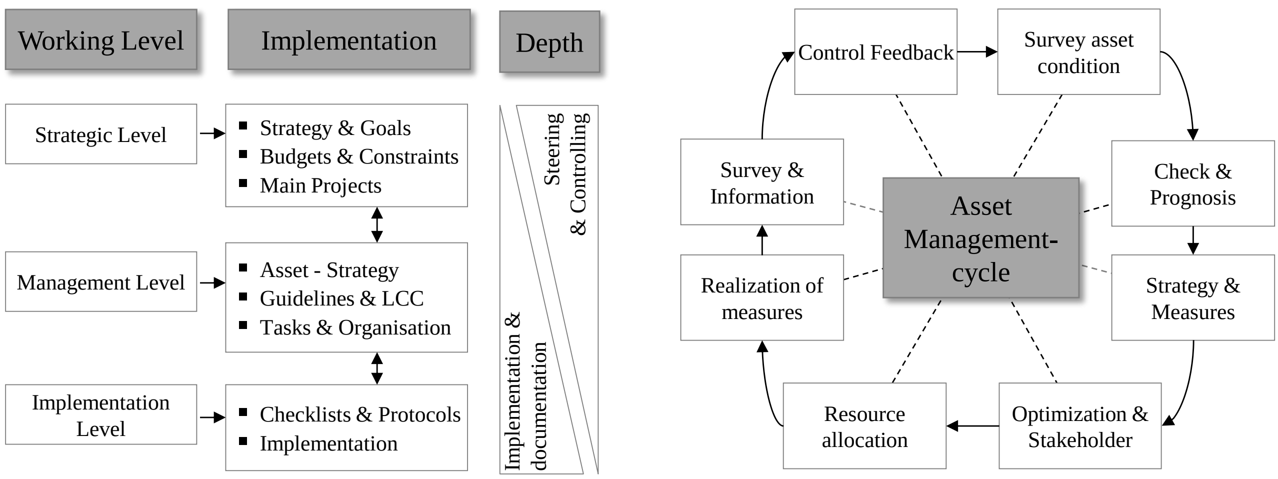

Figure 38: Asset management structure and implementation cycle of tasks [Hoffmann et al. 2014d]

3.2 New asset management approach

The presented waterway asset management approach (Figure 39) accounts for these factors with the *fairway availability of widths and depths* in *days per year* as a core parameter. Each *recommendation or agreement* (e.g. UNECE 1996, Danube Commission 2013) can be modelled as *a single point*. The position of this point in a 3D availability chart is determined by defined fairway widths and depths in addition to the required days of availability. With a 3D data model of the river including *locks*, *ports* and the *fairway* the resulting overall availability includes both *non-availability* and available *fairway widths* and *depths* for *certain levels* in days per year, resulting in a 3D availability surface. River maintenance and engineering works modify this 3D river geometry and thus increase the availability of fairway parameters. With increasing targeted fairway widths and depths, the necessary investments will increase as well, leading to a rising 3D cost surface. Besides deliberate interventions, such as dredging measures, there are further impacts on waterway infrastructure and its availability, which can be affected only to a limited extent. These factors include; *precipitation events* and *resulting floods* as well as *temperature-dependent ice formations*, *extreme fog*, *powerful storms*, *vessel accidents*, *lock failures*, *legal or environmental restrictions*.

On the other hand, an increased availability *will in turn lead to a higher utilization* and, thus, lowered costs of inland navigation, which is described by a *falling 3D cost surface*. The fleet model of the Danube vessel fleet is based on calibration curves of all relevant individual vessels and convoys navigating on the river Danube. For a given draught loaded, the respective vessel utilization can be derived providing the basis for transport cost models. Such a transport cost model enables the calculation of availability-based resulting transport cost savings for different investment strategies and entire transport routes. Based on this approach, cost-efficient measures can be found not only for individual critical sections but for an entire river as well. Furthermore, it is possible to optimize, with regard to recommended fairway parameters, a constrained budget or minimal total costs of inland navigation [Hoffmann, M. & Haselbauer, K. & Blab, R. & Simoner, M. & Dieplinger, K. & Hartl, T. 2014]..

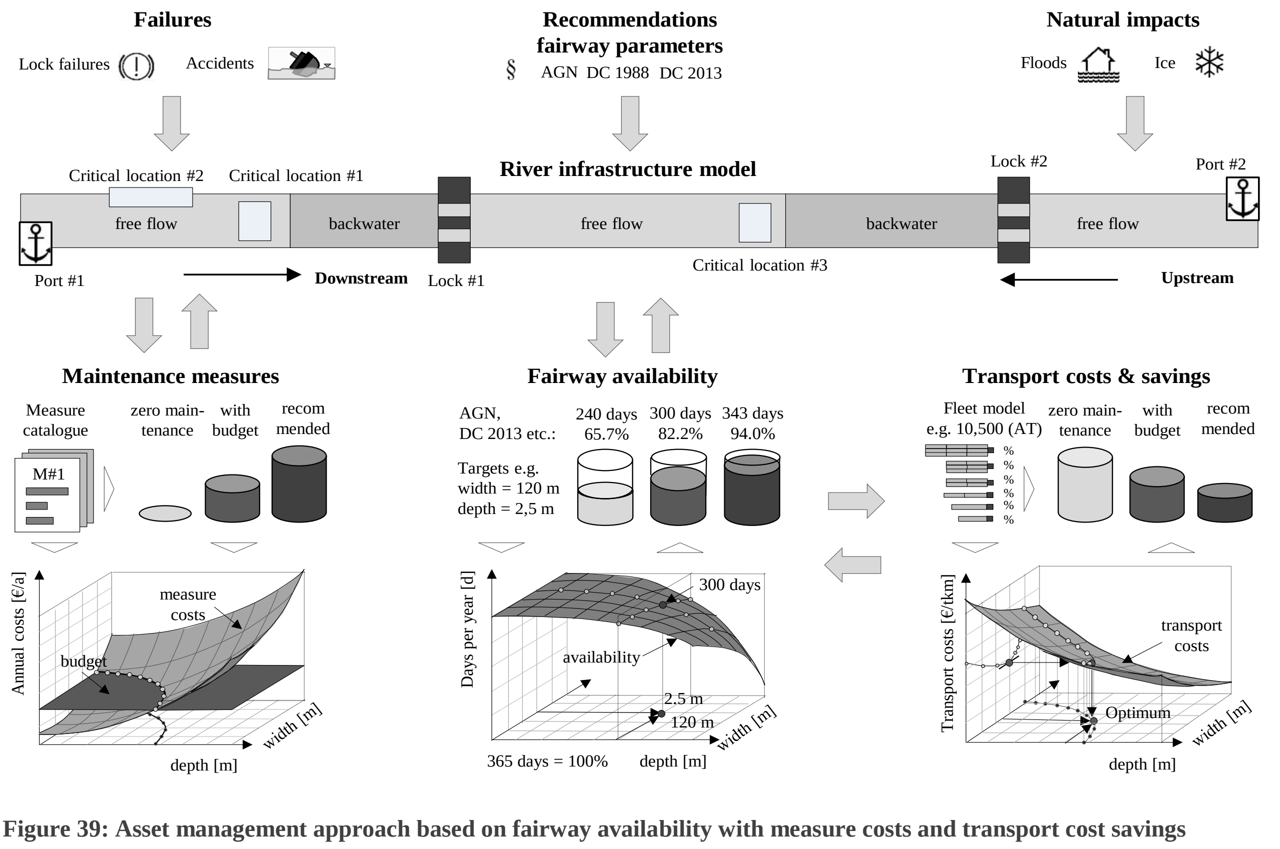

Figure 39: Asset management approach based on fairway availability with measure costs and transport cost savings

3.3 System boundaries – direct and indirect impact parameters on fairway availability

In waterway asset management there are a number of influence parameters mentioned afore, such as floods, low water periods and lock failures, for example, which reduce the availability performance of the fairway channel for inland navigation. But not all influences on fairway availability can be managed within the scope of asset management directly and to the same extent.

In the context of availability-based waterway asset management, however, the risk of negative economic and environmental consequences for stakeholders (especially for navigation companies) due to non-availability or reduced availability of fairway depth is considered an important decision criterion for the derivation of appropriate prevention and mitigation strategies. A classification of impact parameters on fairway availability, therefore, contains the categories *predictability*, *probability of occurrence* and *estimated monetized consequences* for the shipping industry (or environment).

Essential for this availability-based approach are, in particular, the consequences of an influencing event on fairway availability. The *severity of an impact* and the resulting monetization are based on the *extent of width and depth restrictions* and the *number of affected days*. A *total shut-down of navigation* over a longer time period (e.g. a few weeks) is thus considered as extremely serious for navigation.

With decreasing predictability of impact parameters, the demand for hazard mitigation strategies increases, especially for impact parameters with severe consequences for inland navigation. For impact parameters with both a high predictability and probability of occurrence, hazard prevention strategies are applied. In the case of significant consequences for users, these strategies are also often combined with appropriate mitigation strategies. For minor consequential effects, however, accounting for a respective impact parameter, within the scope of waterway asset management, depends on the available budget.

3.3.1 Agreements and recommendations for fairway parameters

Existing agreements and recommendations regarding fairway parameters of waterway of international importance, such as the *European Agreement on Main Inland Waterways of International Importance* (AGN), aim at providing common minimum standards for *Inland Waterway Transport* (IWT), meaning that uniform standards for fairway depth are supposed to be realized on entire transport routes. But with increasing requirements regarding fairway dimensions, the probability that these target fairway depth are available on a given day decreases as well as the cumulative availability of the respective fairway dimensions throughout the year. Due to insufficient budgets, existing agreements in practice represent rather *ambitious objectives* as *uniform minimum standards*. As a result, the Danube waterway, nearly 3000 km navigable in length, shows *strongly varying fairway depths* on respective national river stretches, subsequently leading to *unfavorable effects* on *utilization of the Danube vessel fleet*. Through reducing requirements regarding fairway width and simultaneously guaranteeing a Danube-wide uniform fairway depth, a much more favorable availability performance for inland waterway transport could be achieved within the budget of waterway authorities. The severity of consequences of a reduced availability or non-availability of the fairway depends on the respective characteristics of individual shallow sections and the water level. A shut-down of navigation when the lowest navigable water level is reached may be defined as worst case scenario where the severity for inland navigation increases rapidly with the duration of the period of ineligibility. Thus, in asset management of waterways agreements concerning the availability of fairway dimensions are considered as target performance parameters that may be compared to the actual availability performance of national river sections in the context benchmarking. The fairway dimensions defined by the AGN may only barely be influenced; however non-compliance remains mostly without any consequences for the defaulting waterway administrations. Within the framework of asset management, however, it is possible to derive a minimum standard of fairway depth and width that may be *continuously achieved on the entire transport route with the given, fixed budget*. In the case of sufficient budgets, a measure program may be determined based on an optimization process, which allows *achieving an availability target* (width and depth with availabilty on a certain number of days) with *a minimal use of financial resources*. Moreover, on the basis of a comprehensive waterway asset management approach, it is further feasible to derive uniform fairway widths and depths that may be achieved on the entire Danube river that constitute economically optimal fairway dimensions from both the perspective of waterway administrations and from a customer's perspective as well. Therefore, a new regulation of existing agreements must be based on waterway asset management and would thus represent a first step towards a customer-centric modern waterway management, and also subsequently enhance the competitiveness of inland waterway transport.

3.3.2 Impact of failures and accidents

The availability of waterways is further influenced by failures of facilities that are critical bottlenecks of the waterway transport system such as locks or bridges. In any transport system with a *serial configuration of assets, the failure* of *one single asset* leads to an outage of the entire transport route. In addition to failures, scheduled revisions or major lock repairs may also lead to days with a non-availability of the waterway for inland navigation. On the Danube, all locks are constructed redundantly with two lock chambers, so that in the case of a sudden failure of one lock chamber, inland waterway transport can be further preserved, although some delays must be expected. Due to the given redundancy of lock chambers, the availability of Austrian locks exceeded 99 percent during the resent years [VIADONAU 2015].

In addition, vessel accidents in the central fairway path may result in a long-lasting blockage of navigation. Although sinking of vessels is very rare, large vessel dimensions and a remarkable flow velocity contribute to long salvage maneuvers. Based on annual reports of the Austrian waterway administration VIADONAU, typical accidents on the upper Danube include *groundings, sunken vessels, bridge strikes* as well as *collisions with vessels*, *lock facilities* (most frequent cause) and *riverbanks*. For navigation companies operating on the Danube, vessel accidents represent *only minor obstacles* in practice.

In order to enhance the availability of inland waterways, asset management provides a range of methodological foundations facilitating *decisions under risk*. Hence, there are a number of mitigation and prevention strategies that may be employed in order to *deal with risk-prone impact parameters* [PIANC, 2013]:

- *Reduction of asset failure probability* through implementation of appropriate preventive maintenance measures and inspection techniques.
- *Reduction of failure impact* by increasing asset redundancy and preparation of mitigation measures and disaster control plans.
- *Accepting asset failures,* reduced availability and associated consequential costs.
- *Transferring asset failure risks* by *purchasing insurance policies*.

Concerning the impact of asset failures and vessel accidents on waterway availability, asset management allows for control, especially deterioration based failures obtained by cyclic inspection, maintenance and rehabilitation procedures. Furthermore, failure extent and frequency of vessel accidents may also be addressed by the means of a comprehensive asset management policy, which especially strengthens the awareness of necessary upgrading of the vessel fleet. With decreasing asset failure predictability, the *expensiveness of conventional preventive* and *damage mitigation approaches increases progressively*. Under the assumption of *budget constraints* for waterway operators and customers, both *failures and consequential costs are accepted in practice* if *costs of prevention reach uneconomic dimensions*. When failures additionally show a high risk potential for human safety or entail other severe consequences, *insurance policies covering those failures* are the preferable choice in asset management strategies. This thesis is focused on the discussion of condition-based non-availability of the fairway, which also includes the availability of locks in a broader sense, but lock facilities, however, will only be discussed in general availability calculations of the waterway, and will not be included in the conception of measure programs.

3.3.3 Natural impacts on fairway conditions

For natural modes of transport, such as the Danube waterway, there are also a number of influence parameters with a negative impact on the availability of the fairway that are based on *natural phenomena* such as *high-* and *low water periods*, *fog* and *ice-formations,* which may be controlled only *to a limited extent* with asset management methods. Prolonged periods of cold weather during winter months may lead to increased ice formation in extreme cases, which complicate the navigability of the fairway with increasing ice thickness and finally result in a closure of navigation. For the calculation of fairway availability this means that, *for all days with ice blockades, no availability* is given for all combinations of fairway width and depth. These days with non-availability are then included in the cumulative calculation of overall fairway availability. The temperature profile during winter months cannot be influenced by waterway operators or navigation companies. By the use of icebreakers, waterway operators have the opportunity to mitigate ice formations. However, this alone may not be sufficient to avoid interference of navigation, since all assets associated with inland waterway transport as well as necessary equipment, such as vessels and convoys themselves, are stressed by ice.

Fog as a visible mass consisting of cloud water droplets or ice crystals suspended in the air, is often considered as a type of low-lying cloud, and is heavily influenced by *nearby bodies of water, topography, wind conditions, and human activities*. Strong *fog formations* reduce the visibility of obstacles and other vessels. For inland navigation fog indicates that travelling speed must be reduced, and the importance of navigation-technical tools increases. Foggy conditions, however, are mostly a *temporary natural phenomenon*, so although there are delays that affect transport costs, only seldom are total blockades of navigation and consequently non-availability of the fairway, a result of fog. Therefore, fog as an influence parameter in asset management is considered only *a posteriori* in the calculation of availability if limitations and barriers for navigation occur. Through navigation, supporting technologies such as radar, restrictions for navigation, however, can be largely avoided.

High water levels[26] occur with a *certain probability*. Based on water level data collected at gauges within recent decades, *water level lines* and the implications of different flood events are known and well documented. Therefore, annual peak values are selected from long-term measuring series in a first step, followed by the *calculation of exceedance probability*. The reciprocal value of this exceedance probability is termed as annuality and refers to the *statistical recurrence interval*. In hydrography, the discharge volume is abbreviated with “Q” (from lat. quantitas) and high water levels with “H”. The notation “HQ” therefore refers to a flood event occurring statistically every 100 years, also call flood of the century. Through rising

[26] *Also see https://de.wikipedia.org/wiki/Hochwasser; (Last access: 07.09.2015)*

water levels, vertical clearances under bridges are no longer sufficient for navigation and locks have to be closed for safety reasons. Therefore, inland navigation is shut-down, at least when the *statistical highest navigable water level* is reached, resulting in a *non-availability of the fairway*. Blockades obviously arise also for even *higher flow rates than HQ30* (occurring every 30 years) *or HQ100* (occurring every 100 years). Flood events and their impact on fairway availability are easy to include in availability calculations within waterway asset management. Waterway asset management tools allow a backwards evaluation of the impact of flood events on river engineering structures, shore constructions and the riverbed. However, a forecast of the precise occurrence timing is not possible even with the methods of asset management. Flood protection management is not a traditional core task in waterway asset management yet, but expansions towards flood management are feasible as well as the creation of interfaces to flood protection tools. However, by using occurrence probability together with the *evaluation of the impacts of flood events*, budget reserves may be generated, which may in turn be used for the implementation of mitigation measures. In addition, preventive measures, such as the construction of flood channels and retention basins, may be planned and implemented based on the methods of asset management.

For inland waterways, *low water levels* are primarily a result of *meteorologically induced lacks* of *precipitation*. The appearance of low water events depends on local climate (discharge regime). Thus, low water level may occur during summer time, but also during winter, as in mountain regions where precipitation is bounded as snow or ice. Low water events may be defined based on *statistical thresholds* regarding different viewpoints in ecology, water management or navigation. Low water periods for inland navigation are linked to statistical definitions. Thus, the *low navigable water level* (LNWL) is defined as a statistical water level which is exceeded on 94 percent of days of a year [VIADONAU 2013 b]. Low water periods announce themselves by *longer periods of low rainfall* and lead to reduced fairway depths and, hence, reduced possible draughts and a low utilization of the vessel fleet. Depending on the riverbed geometry, shallow sections appear as lateral or centered obstacles in the fairway. With increasing fairway width, the probability of a full available fairway depth on the entire fairway width decreases rapidly. In the long run, the competitiveness of inland navigation suffers increasingly with increasing duration and frequency of low water periods. Based on forecasts of riverbed and water level developments, the influence of low water on fairway availability may easily be controlled with asset management systems. This also includes planning and implantation of preventive measures. Depending on the fairway geometry, even during low water periods some areas of the fairway with sufficient depths may remain, allowing narrowing and, in turn, re-marking of the new fairway path so that navigation is further possible on this river section without any depth restrictions. As low water events *often occur seasonally*, appropriate coordinated and proactive dredging measures reduce negative impacts on inland navigation. The efficient allocation of budgetary resources represents a core task of availability based waterway asset management.

A further impact parameter on the fairway availability is closely related to precipitation events and commonly known as erosion. Erosion of rock and gravel from mountain regions, shore constructions, river engineering structures and river bed material, on the one hand, leads to deepening tendencies of the river bed and consequently results in numerous sedimentations, which may grow to shallow sections in the course of time. The transport of sediment is currently only indirectly included in the empirical riverbed model based on riverbed surveys and must be extended to periodic measurements of bed load and fine sediment, as well as numerical approaches and field testing, in order to provide a comprehensive management of this impact parameter on fairway availability.

4 CONDITION MODELING IN WATERWAY ASSET MANAGMENT

4.1 Basic model of waterlevels, riverbed and fairway parameters

Forming the basis for evaluations of the availability of inland waterways, the main components of the infrastructure model are *lock chambers*, *ports*, as well as *numerous transshipment sites*, *berths* and *landing stages* with widenings of the fairway. These assets affect the resulting availability of the transport route (e.g. due to unexpected lock failures or planned maintenance works of lock chambers). Further negative impacts on availability may also arise in the form of *lowered accessibility of port facilities* due to *sedimentation processes* (e.g. in the area of port entrances). For a realistic picture of present and past availability conditions, it is therefore crucial to take all of these aspects into account.

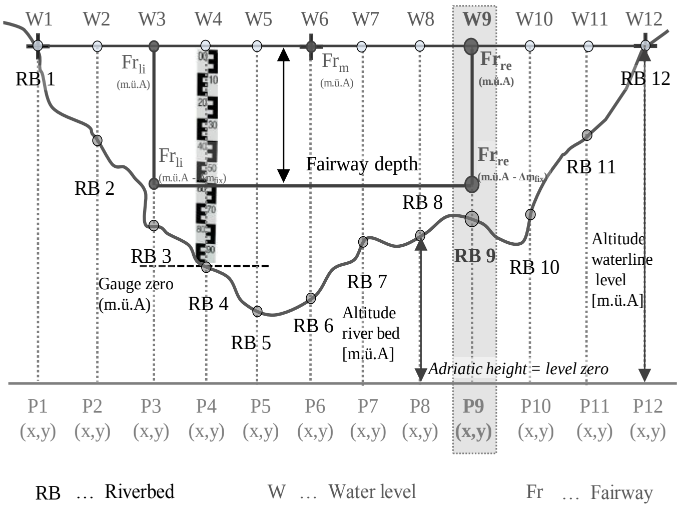

Figure 40: Model of a cross-sectional profile of a river section including current water level, riverbed morphology and fairway parameters in absolute altitudes

The presented approach is substantially based on a model of *free-flowing* and *backwater sections* consisting of 3D data of *riverbed* and *water level*, and their *changes in the course of time*. The information is given by coordinates describing the position on a horizontal level (x, y) and the absolute altitude of the respective points (Figure 40). The fairway is linked to the current water level line and can be modeled for each recommended fairway width and depth. The calculated availability is the result of combining *fairway classes* with changes of the *riverbed and water level* on a daily basis *equaling a 3D surface*. Other important reference levels, such as the *low navigable water level* (LNWL), may be determined based on a statistical analysis. One advantage of using absolute height data compared to such a statistical value is to enable the mapping of *over-deepening tendencies* of the riverbed. For an integration of the entire Danube River in such a system, the different *national coordinate systems* and *altitude references* will have to be harmonized into one unified database [Haselbauer, K. et al. 2014].

If availability is calculated on the basis of cross-sectional profiles, the necessary density of riverbed surveys depends on riverbed characteristics. *Critical sections* (i.e. shallow and/or narrow) that are relevant for availability *require a higher density* compared to non-critical sections. For critical sections, additional multi-beam surveys are regularly used in order to get a more accurate picture of the development of the riverbed between periodic standard single-beam surveys. These multi-beam data can be thinned out by using various algorithms in order to avoid excessive amounts of data. Single- or multi-beam data are the basis for the calculation of *riverbed isolines* that are used to display riverbed and *current fairway conditions*. As a result of continuous riverbed surveying and data modeling, an analysis of sedimentation and erosion processes is possible as well (e.g. with *difference maps*) between two riverbed surveys as possible results. Furthermore, navigational charts with fairway depths, in relation to low navigable water level (LNWL) can be prepared as well. For the availability-based waterway asset management approach a model of the water level is needed. This water level can be modeled based on various methods. The actual water surface can be calculated by a simple linear interpolation between current water levels.

4.2 Water level model

In order to implement a *water level model* for entire river sections which also accounts for different flow scenarios, *a network of monitoring stations* is required. These monitoring devices are also referred to as *water level gauges*. Depending on the construction type and available equipment, further parameters, such as *flow rate* (Q) or *water temperature* (T), may be measured in addition to the *current water level* (W). Some gauging stations are highly automated and may include telemetry capability transmitted to a central data logging facility. Gauging stations are closely related to the local hydro-morphological characteristics and generally *have to be situated* at locations *with significant changes in the hydraulics* of the riverbed. Thus, the number and density of gauging stations necessary to achieve a sufficient accuracy of water level models depend inter alia on the length of the respective river stretch, the *gradient of the riverbed*, the number and location of *feeding rivers*, and the number and location of *power plants* which may result in a sudden and massive increase of the water level. *Important gauging stations* are situated in *backwater a*nd *downstream* of power plants. They are particularly important because the water levels downstream can *vary substantially*, as they are mostly controlled by power plant operators which are trying to *serve actual energy demands*. According to Boiten[27], the position of a gauging station as a part of a monitoring network should be chosen in a way so that water level information can be provided for each point cross section or river point, at least by the means of interpolation. The design of the measuring network must allow a comprehensive reproduction of all feasible water level variations, reaching from extremely low water levels up to extreme flood events [Morgenschweis, G. 2010]

4.2.1 Basic hydrological data requirements for water level modelling

At the location of gauging stations which are generally situated on the river banks, the accurate altitude of the water level is measured, and may be determined, as absolute altitude above any given reference level or as relative height above the respective gauge zero. The equipment of a gauging station may vary considerably; it ranges from *discontinuously operation staff gauges*, *self-registering systems with mechanical recording* of water levels up to *electronic systems with digital storage* of water level data. In order to outline the measuring principle of non-self-registering water level measuring systems, definitions of absolute altitude, reference water level, and gauge zero are illustrated in Figure 41 for a common staff gauge. The most common self-registering gauging devices include pressure sensor gauges, floating gauges and ultrasonic sonar gauges [Morgenschweis; G. 2010].

For inclined staff gauges with given length l and slope angle α, the actual water level h can be calculated as follows:

$$h = \frac{l}{\sqrt{1 + \frac{1}{tan\ \alpha^2}}}$$

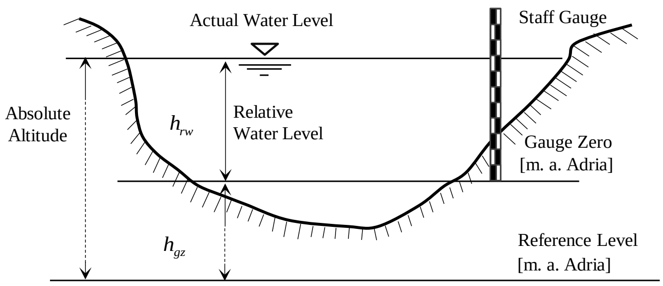

Figure 41: Inclined staff gauge river Danube [Manual on Danube Navigation] and Definition of absolute altitude, gauge zero and actual water level for vertical staff gauges (unscaled illustration of a cross section)

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[27] *Boiten, W.: Hydrometry. CRC Press/Balkena: London, 2008 (3rd edition)*

The *relative water level height* (h_{rw}) is defined as relative vertical distance of a point of the watersurface, which may be above or below a *reference horizon* (gauge zero). The height of the *absolute water level altitude* (h_w) is usually specified in meters and can be calculated based on equation (38). For the *reference level gauge zero* (h_{gz}), the absolute altitude above a reference sea level, such as Adriatic Sea level, is known.

$$h_w = h_{gz} + h_{rw} \qquad \text{in m. a. A.} \tag{38}$$

Due to different vertical locations of the respective gauge zero, absolute altitudes based on a common reference sea level are often used to describe the water level line. In general, gauge heights are measured several times a day. Measured water levels alone, however, do not provide any information about current fairway depths. This is due to the fact that the gauge zero, (i.e. the lower end of a gauge staff or altitude of a gauge), does not correspond with the location of the riverbed. The gauge zero can lie above or below the medium riverbed level of a river section. Therefore, surveys of the current riverbed altitude are necessary in addition to water level gauges.

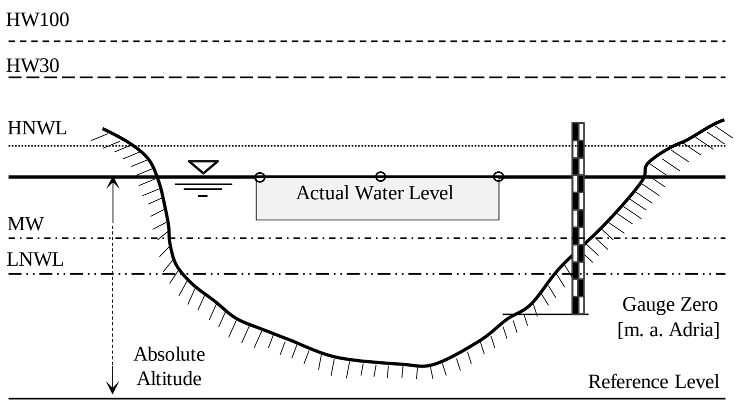
Due to the length of the Danube River and the number of the riparian countries, various reference water levels corresponding to different sea levels are used on this river. Thereby, the mean sea level measured at a *gauging site of the nearest ocean coasts* serves as the reference for determining the *absolute or geographic level of a gauge zero on the earth's surface*, the so-called *zero point*. As mentioned above, the water gauges along the Danube have different reference points, for example the North Sea (Germany), the Adriatic Sea (Austria, Serbia), the Baltic Sea (Slovakia, Hungary) and the Black Sea (Bulgaria, Rumania, Moldova & Ukraine). As the water level at a gauge changes continually, reference water levels are important to assess the changes on the maintained fairway depth [Manual on Danube Navigation, 2013].

By continuous measurement of water levels, long-term data sets can be generated for each gauge allowing the calculation of *statistical reference water levels*, which serve as important information for a conceivable operation of the fairway and also provide the basic input parameters for *dimensioning of river engineering structures*. Figure 42 provides an overview of a cross section including all important statistical water levels for waterway asset management as *low navigable water level* (LNWL), *mean-water level* (MW), *highest navigable water level* (HNWL), *30-yearly high water level* (HW_{30}) and the *100-yearly high water level* (HW_{100}). These statistical values are of significant importance for the calculation of water levels for any given location and, therefore, defined in detail hereinafter [Manual on Danube Navigation, 2013]:

- **LNWL**
  According to the *Manual on Danube Navigation*, which was published 2013, the low navigable water level at a gauge is defined as *water level that is exceeded* on an *average of 94% of the days in a year* (i.e. on 343 days), excluding periods of ice. For the calculation of this statistical value, water level data collected over a long time period (stretching across 30 years) are used. LNWL also serves as reference level for the determination of fairway depth. In order to keep fairway at a constant minimum depth, conservational dredging measures or groins are implemented, with LNWL being used as reference water level for the construction or implementation of dredging measures. LNWL is further used to ensure uniform illustrations of fairway depths for river stretches that are based on riverbed surveys visualized by hydrographic maps of fairway depths.
- **MW**
  The *mean-water level* is defined as *average water level* at a gauge measured over several years.

- **HNWL**
  According to the Danube Commission the *highest navigable water level* at a gauge is defined as *water level exceeded on an average of 1% of the days in a year*, excluding periods of ice. If the highest navigable water level is exceeded over a certain degree, the responsible authority for this river section may impose a temporary suspension of navigation for reasons of traffic safety. For availability calculations, days with water levels exceeding HNWL are considered as days with a total non-availability of the fairway due to *too low vertical clearance below bridges* and *closure of locks*.
- **HW_{30}**
  HW_{30} is defined as reference water level (high water level) that *corresponds to a discharge* that *occurs at a gauge statistically every 30 years*. High water level lines are particularly important for the determination of flood zones, especially when flood management is concerned, as an integral task of waterway asset management.
- **HW_{100}**
  This high water level corresponds to a discharge that *occurs at a gauge statistically every 100 years*, and is also referred to as *100-yearly flood*. If, in addition to riverbed surveys of the fairway, laser scans of forelands are integrated in a comprehensive waterway asset management database, flood maps or flood control facilities may also be managed.

Figure 42: Cross section layout providing an example of actual water level altitude as well as statistical water (unscaled example illustration of a cross section).



4.2.2 Methods in water level modelling

In order to provide information on the current water level at any given point between two gauges, the development and implementation of water level models is required. Depending on the purpose of water level calculation, one-dimensional, two-dimensional or three-dimensional models may be applied for flow calculation. These models represent a mathematical simulation of the flow behavior of limited sections of natural rivers. Therefore, the definition of system boundaries for the model area is essential for the selection of a calculation method. The models differ regarding the type of concerned flow components (longitudinal, horizontal and vertical) as well as requirements and amount of input data.

Figure 43 provides an overview of basic flow characteristics of *hydrodynamic numerical models* (HN-models) that will be discussed briefly regarding their applicability in a waterway management system in the following sections of this thesis. The term *hydrodynamic* refers to *steady and unsteady flow processes* in *open channels*. Analytically solvable equations are given especially for linear modeling of simple geometric systems. The nonlinearity that often occurs in practice requires the application of numerical models which are based on a variety of system assumptions that are necessary to describe complex nonlinear process equations in a discrete form [State Institute for Environmental Protection Baden Württemberg, 2003].

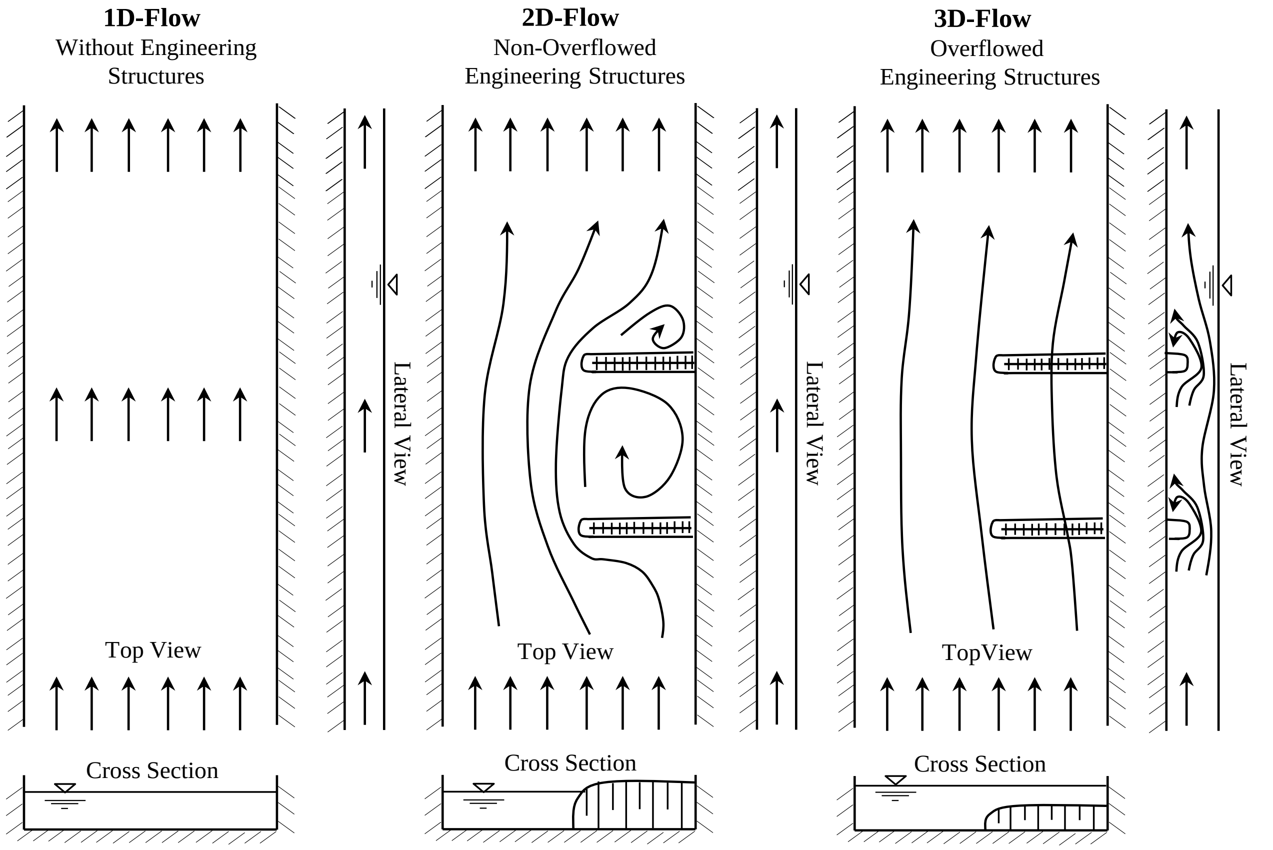


Figure 43: Comparison of one-dimensional, two-dimensional and three-dimensional flow characteristics including non-overflowed engineering structures (2D-flow) and overflowed engineering structures (3D-flow) [Musall, M. 2011]

The *proportions of geometric scales in a body of water* (z << y ≤ x) mark the integration limits of HN-models as coordinates in the top view, cross section and longitudinal section. Thus, model equations of a *lower dimensionality* can be determined based on *integration of 3D basic equations transversely* to the *main flow direction.* For 3D-flow calculations, full *Navier-Stokes equations* are usually applied. Based on these equations, *fluid movements* may be described accurately. A direct solution of these equations is described as very complex, as the *whole spectrum of turbulence* has to be considered. By *splitting up the velocity vector* in a *mean component* and a *turbulent fluctuation quantity*, it is possible to convert the equations into the so-called *Reynolds equations* [State Institute for Environmental Protection Baden Württemberg, 2003]:

u_i	… velocity component in coordinate direction	υ	… kinematic viscosity
x_i	… direction of flow component	τ_i	… shear stress
i	… possible components	Ω	… coefficient of turbulence losses
ρ	… fluid density	g	… gravitational acceleration
p	… hydrostatic pressure	t	… time

$$\frac{\partial u_i}{\partial x_i} = 0 \tag{39}$$

$$\frac{\partial u_1}{\partial t} + u_i \cdot \frac{\partial u_1}{\partial x_i} = -g \cdot \frac{\partial z_s}{\partial x_1} + \Omega u_2 + \frac{\partial}{\partial x_i} \cdot \left(\frac{\upsilon \partial u_1}{\partial x_i} + \frac{\tau_{1i}}{\rho} \right) \tag{40}$$

$$\frac{\partial u_2}{\partial t} + u_i \cdot \frac{\partial u_2}{\partial x_i} = -g \cdot \frac{\partial z_s}{\partial x_2} + \Omega u_1 + \frac{\partial}{\partial x_i} \cdot \left(\frac{\upsilon \partial u_2}{\partial x_i} + \frac{\tau_{2i}}{\rho} \right) \tag{41}$$

$$\frac{\partial p}{\partial x_3} = -p \cdot g \tag{42}$$

Based on three-dimensional model specifications, 1D, 2D and 3D considerations are discussed in the following section together with their simplifications and implementation in hydraulic engineering. Furthermore, the consequences of these model assumptions and their explanatory value are examined regarding their applicability in waterway asset management. Figure 44 provides categorized, basic equations in hydraulic engineering, including their basic model assumptions.

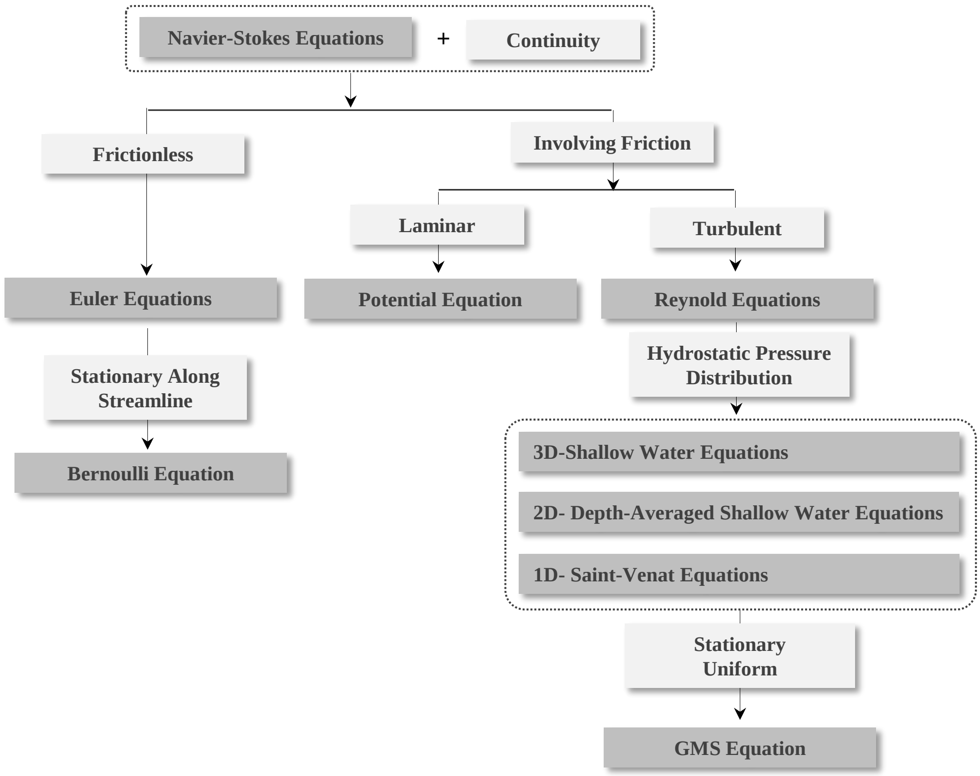


Figure 44: Classification of basic equations in hydraulic engineering [Musall M. 2011] and [Rutschmann 2003]

The described 3D-Renoynolds equations may be further simplified by the *negligence of vertical impulse forces*, subsequently leading to so termed *3D-shallow water equations*. These may be applied for water bodies *with a great length* and *width expansion in relation to water depths* (with a ratio length/ width to depth ≥ 10/1) as, for example, flood plains [Musall, M. 2001]. 3D-models provide more comprehensive information than 2D-models as a more detailed calculation of velocity distributions close to the riverbed is feasible, serving as prerequisite for an accurate transport simulation for sedimentation and erosion processes.

Because of given geometric flow proportions (z << y,x), a further simplification of Reynolds equations is possible by *mathematical integration over the water depth*. This *averaging of depth* results in *2D-shallow-water equations*. These *shallow water equations are commonly used in practice*. They especially allow an accurate calculation of flow characteristics for complex flow conditions or riverbed geometries with *engineering structures* and *flooded forelands with rich vegetation*.

1D-model equations may be applied for river sections with predominantly *one-dimensional flow characteristics* which are described, for example, by a *moderate curvature* and *regular cross sections*. *Averaged water levels* and *flow velocities* for a cross-section can be generated as a result. In order to take into account *flow changes* in *cross-flow direction*, which typically occur during flood periods in wide floodplains, *1D-models for structured cross sections* have been developed. The derivation of 1D-model-equations is carried out by *vertical and horizontal integration of 3D-Reynolds equations*. The so termed *Saint-Venant equations* are based on the simplifying assumption that *all velocity components*, *transverse to the main flow direction*, *are negligible*. The river is thus described as s single flow canal showing a horizontal surface transverse to the flow direction and a gradually varying cross section [State Institute for Environmental Protection Baden Württemberg 2003]. For each of these HN-model calculations, a number of input parameters are required, where extent and complexity of input data is generally increasing with model dimension:

- *Topographic data* of river/channel morphology (digital riverbed and terrain models)
- *Hydrological data* (measured water levels, discharge data and occurrence probabilities)
- Data on *friction slope* and vegetation (based on orthophotos)

Depending on individual circumstances there are a number of criteria which allow an informed selection of an appropriate mathematical model for water level calculation:

- *Flow characteristics* of the model area (mostly 1D or 2D-, 3D flow conditions)
- *Target parameters* of the HN-calculation (water level and/or flow velocity, discharge)
- *Dimension*/size of the model region
- Existing basic data (topographic, hydrological, flood protection constructions)

Thus, 1D-flow models only take velocity components aligned with the main flow direction into account, and can be very well applied for simple cross sections without river engineering structures (compare Figure 43). If the flow characteristic of a modeled river section can be describes as predominantly one-dimensional, average water levels and flow velocities can be calculated as a result of the application of 1D-flow models.

Furthermore, simplified iterative calculations of water levels may also be applied. Thus, emanating from an initial cross-section, the water levels that appear for slightly non-uniform discharge conditions may be calculated based on the *Gauckler-Manning-Strickler-equation* (GMS). The water level calculation starts at an initial cross-section where the flow conditions are known. The necessary input parameters and the equation itself are described in formula (43):

$$v_m = \frac{k}{n} \cdot D_h^{\frac{2}{3}} \cdot S_f^{\frac{1}{2}} \qquad D_h = \frac{A_i}{P_{w_i}} \qquad v_m = \frac{Q}{A_i} \tag{43}$$

v_m	… average flow velocity [m/s]	S_f	… friction slope [m/m]
D_h	… hydraulic diameter [m]	A_i	… cross-sectional area [m²]
n	… Gauckler-Manning-Strickler coefficient i.e. 33-35 $\left[s/m^{\frac{1}{3}}\right]$	P_{Wi}	… wetted perimeter [m]
k	… conversion SI i.e. k=1		

For a second neighboring cross section with unknown flow conditions being located in longitudinal direction at the position *L*, the water depth h_2 is estimated as a first guess. Based on this assumption, a calculation of the hydraulic diameter and the flow velocity for the section are possible. Subsequently, also their mean values between these two sections may be derived as well. By substituting these values in the *energy balance equation of Bernoulli*, the water level difference Δ*h* may be calculated as target size. For the case that Δh is equal to h_2-h_1 the sought water level for cross section 2 is found, and the iteration process may be continued with the next longitudinal interval. This iteration can easily be used for water level calculations in simple cross sections and applied in the context of asset management based on ordinary Excel-tools [State Institute for Environmental Protection Baden Württemberg, 2002].

For *water level accuracy*, required for the *purpose of inland navigation*, the *application of 1D-water level models* is *unquestioned* among experts. For one-dimensional flow calculation, a number of common software packages, such as HEC-RAS[28], MIKE11[29] and WSPWIN[30], are available, among others. Figure 45 provides an overview of 1D flow modelling using HEC-RAS software. HEC-RAS, as complimentary and traditional software, offers the advantage that a lot of experience, regarding accuracy and applicability, is already available among experts. As a part of studies conducted for the implementation of an Austrian Waterway Asset Management System, a comparison of the accuracy of calculations for water level elevation due to groins HEC-RAS was applied and compared to the results of the 2D-HN software solution Hydro_AS-2D[31] that is commonly used in Austria [Krouzecky, N. et al. 2015].

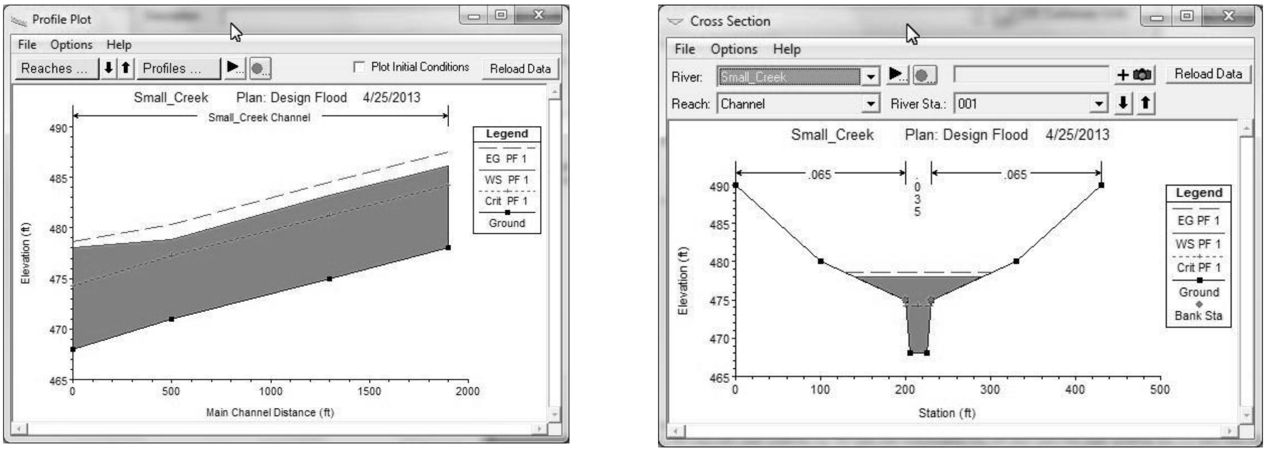


Figure 45: 1D-water level modelling with HEC-RAS; http://user.engineering.uiowa.edu/~water/ras/RAS_work_2013.htm.

2D-water level models are, in practice, implemented in the case of non-overflowed river engineering structures, such as groins with horizontal velocity components occurring, leading to either sedimentation or erosion processes depending on the inclination of the groin axis. For overflowed river engineering structures, such as groins (illustrated in Figure 43), flows include significant vertical velocity components that have to be considered for 3D-HN calculations. Ahead of the development of the Austrian WAMS, different software tools for 1D, 2D and 3D water level modeling approaches were compared regarding their accuracy, data-demand, possible interfaces and possible implementation in a waterway asset management system. In this context, the calculation of water levels with HYDRO_AS-2D was examined for a partial pre-calculation of statistical water level lines by the Austrian waterway agency VIADONAU. The software is based on the numerical solution of shallow water equations using finite-volume-discretization and was also applied in the comparison of groin impacts on the water level. For 2D-HN analysis, common model assumptions for river profiles based only on cross sections are, in most cases, not sufficient enough for 2D-flow calculation. Therefore, own grid generators that intend to compute the actual riverbed, based on meshing, as accurately as pos-

28 *http://www.hec.usace.army.mil/software/hec-ras/; (Last access:15.08.2015)*
29 *http://www.mikepoweredbydhi.com/products/mike-11; (Last access: 15.08.2015)*
30 *http://web.bjoernsen.de/manual/index.php/WspWin/dataformats/de; (Last access: 15.08.2015)*
31 *http://www.aquaveo.com/software/sms-hydro-as-2d; (Last access: 15.08.2015)*

sible for the respective calculation. Methods of riverbed modelling will be discussed later on in chapter 4.3. Figure 47 provides an example of the application of HYDRO_AS-2D for mesh-generation in a river section, including locks as well as calculated flow lines. Further software applications used for 2D-model simulation include HydroSim[32], TELEMAC[33] and CFX[34].

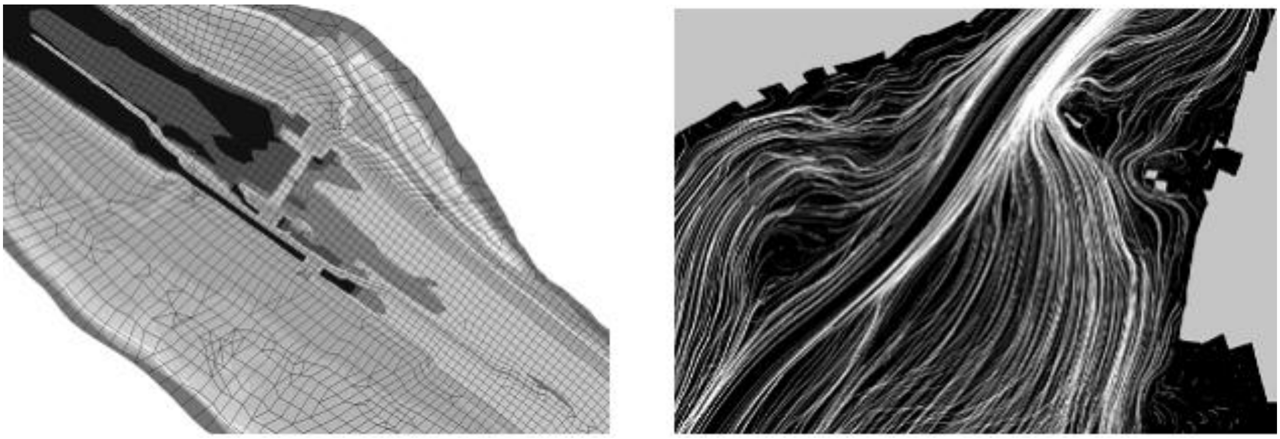


Figure 47: Example of 2D flow simulation with HYDRO_AS_2D showing mesh-generation and flow lines.

As aforementioned, 3D-HN-calculations require a huge amount of input data, consequently resulting in extremely long computing times, so that only very small model regions are suitable for such calculations. Although there is a tendency to enlarge the model area, actual computing capacities remain an almost insurmountable restriction. In practice, software solutions, such as FLOW-3D35 (Figure 57) and SSIIM36, are commonly used for 3D-HN modelling. In contrast to 2D-models and 1D-models, flow variables are not averaged in depth or in the cross section. As the effort to create a 3D-model which includes mesh generation, generation of initial and boundary conditions, parameter determination, computation and visualization of resulting three-dimensional data is enormous, and a reflection considering whether the initial problem justifies this effort is indispensable. As a result, 3D-models are not applied for water level calculations of entire river sections. In order to meet the requirements of inland navigation regarding the accuracy of water levels and fairway depths, 3D-models are too far beyond the objectives.

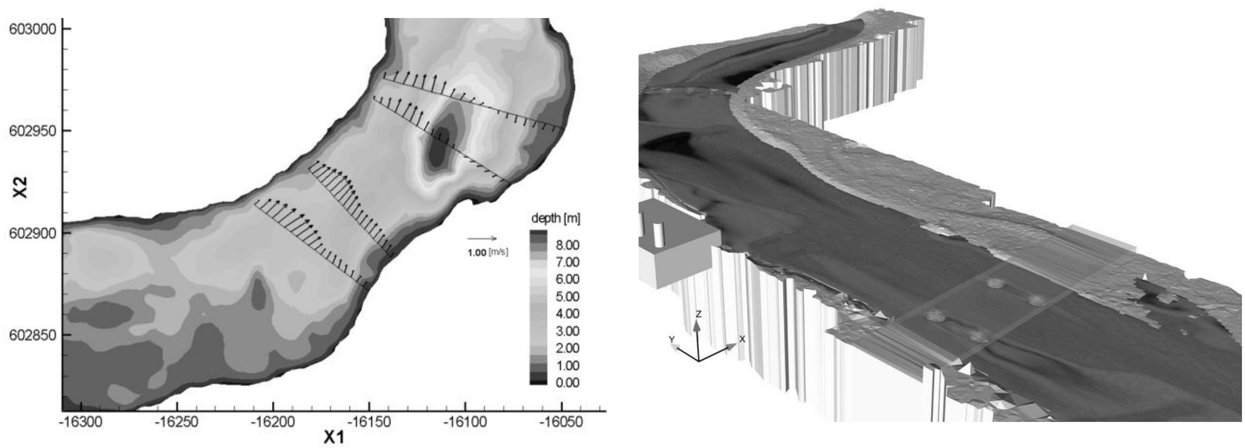


Figure 46: Typical applications of Flow3D- showing computed velocities in some cross sections and high flow condition hydraulic analysis [Flow science: http://www.flow3d.com/home/industries/water-environmental/rivers; 15.08.2015)]

[32] *http://www.hydrosim.de/;(Last access:15.08.2015)*
[33] *http://www.opentelemac.org/index.php/presentation?id=17; (Last access:15.08.2015)*
[34] *http://www.ansys.com/Products/Simulation+Technology/Fluid+Dynamics; (Last access:15.08.2015)*
[35] *http://www.flow3d.com/home/industries/water-environmental/rivers; (Last access:15.08.2015)*
[36] *http://folk.ntnu.no/nilsol/ssiim/; (Last access:15.08.2015)*

4.2.3 Applied water level method

The Austrian free-flowing sections of the Danube River in the area of Wachau (from Krems rkm 2000 to power plant Melk rkm 2038.2) and east of Vienna (from the border to Slovakia rkm 1975 to power plant Freudenau rkm 1921.1) are characterized by a very dense network of gauging stations. Therefore, calculations of water level lines and evaluations of fairway depths for navigation purposes, carried out using the Austrian Waterway Asset Management System, are based on the methods of linear interpolation. However, it is also possible to integrate the results of any 1D, 2D or 3D HN-model via flexible interfaces and a respective database structure.

Depending on the purpose of evaluation, different methods of water level calculations at cross section profiles can be applied. The cross sections at the Austrian Danube stretch show a longitudinal distance of 25 meters in the area of free-flowing sections. One option to calculate water level for cross sections between two gauging stations, is a direct linear interpolation between the current water levels at these gauging stations (see Figure 50). For cross sections that are situated in the periphery of the gauging network (e.g. in border regions with neighboring countries), the method of linear extrapolation is used for water level calculation. A prerequisite for the application of linear interpolation between two gauging stations is given by an analysis of the importance of individual gauging stations, in terms of accuracy of water level calculation. Subsequently a classification of water level gauges is required, allowing a clear distinction between gauges that only lead to an insignificant improvement of water level information and those which are really essential for the performance of the model. Thus, a failure of some gauging stations may also be compensated. If a malfunction of an important water level gauge occurs, the calculation of water levels for arbitrary cross sections is still possible, since the interpolation process in WAMS may also be based on statistical reference water level lines (illustrated in Figure 51) that have been pre-computed by 2D-HN programs (compare calculation Figure 52).

In numerical analysis, interpolation[37] is a method of constructing new data points within the range of a discrete set of known data points. One of the simplest methods is linear interpolation, and generally describes a type of approximation process. The interpolation function, thereby, exactly describes the target function at the respective interpolation points and represents the remaining points approximately. The accuracy of approximation depends on the selected interpolation method (linear, polynomial, spline, trigonometric, multivariate, bilinear, bicubic, trilinear or Gaussian process) and the characteristics of basic data. In contrast to other methods, such as regression analysis, interpolation assumes that the target function proceeds exactly through the given data points.

Extrapolation[38], however, is used for the estimation of values of the target function with data point exceeding the domain of available data points, and thus is subject to greater uncertainty. Based on a single reference point (given water level at the last gauging station) and a given mathematical function, the required water level at any cross section may be estimated. A sound choice of which extrapolation method to apply relies on prior knowledge of the process that created the existing data points. Extrapolation means creating a tangent line at the end of the known data and extending it beyond that limit. Linear extrapolation will only provide sound results when used to extend the graph of an approximately linear function or not too far beyond the known data. Linear extrapolation in WAMS is used for example in border areas with neighboring countries, if the water levels of the next gauging station are unknown.

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[37] *https://en.wikipedia.org/wiki/Interpolation; (Last access: 15.082015)*
[38] *https://de.wikipedia.org/wiki/Extrapolation; (Last access: 15.082015)*

The method of linear interpolation dates back to Isaac Newton and is used most commonly in practice. Figure 48 illustrates the principle of linear interpolation between two gauging stations, representing the most important option of water level calculation in WAMS. Given water levels (y_{G1}, y_{G2}) at gauging stations (G_1,G_2) are thereby connected linearly in order to calculate the water level at any given cross section (CS_i) between these gauges using equation (44):

$$y_{CS_i}(x) = y_{G1} + (y_{G2} - y_{G1}) \cdot \frac{x_{CS_i} - x_{G1}}{x_{G2} - x_{G1}} \tag{44}$$

$G_{1,2}$ … gauging stations
CS_i … cross-sectional profile i between two gauges
CS_e … cross-sectional profile e outside measuring area
$x_{G1,2}$ … kilometrage of gauging station
$y_{G1,2}$ … water level at gauging station [m.a.A]
x_{CSi} … kilometrage of cross-sectional profile i
y_{CSi} … unknown water level at cross-sectional profile i
x_{CSe} … kilometrage of cross-sectional profile e
y_{CSi} … unknown water level at cross-sectional profile e

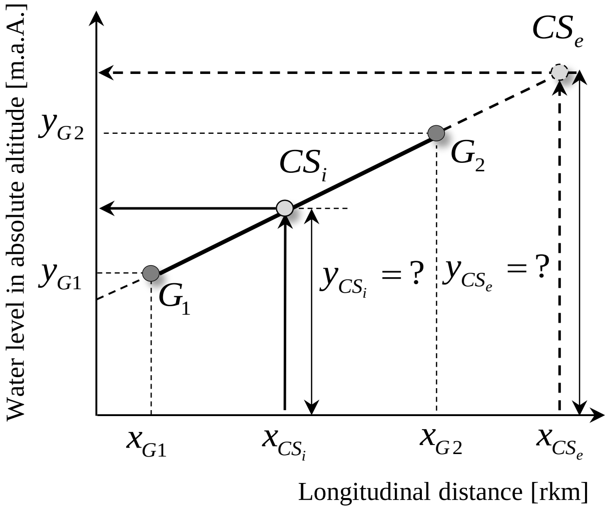


Figure 48: Principe of calculation of water levels at cross sections with given longitudinal distances using linear interpolation

The following section describes the calculation of the current water level for a selected cross-section, both by a direct linear interpolation process and by linear interpolation, based on pre-calculated reference water level lines. Figure 49 provides an overview of the free-flowing river stretch of the Danube, including water level gauges in Loiben (rkm 2005.99) and Krems-Stein (rkm 2002.7) as well as a randomly selected cross-sectional profile at rkm 2003.95. The longitudinal distance between the gauging stations amounts to 3.29 km. Both gauging stations provide current information on water levels that is required for the entire river stretch in order to calculate actual fairway depths. Thus, the water level for each point along the fairway axis or for each cross section has to be calculated.

In the given example, the water level in absolute altitudes should be calculated at cross section rkm 2003.95 for a certain date (14.12.2014). The selected cross section is located at a horizontal distance of 2.04km from gauging station Loiben and 1.25km from gauging station Krems-Stein. Method 1 (direct linear interpolation between two gauges) is illustrated in Figure 50. When applying equation (44), the resulting water level for the selected cross section amounts to 193.91 meters above the Adriatic Sea level.

Method 2 (linear interpolation using pre-calculated relevant water levels) is presented in Figure 52. The relevant water level lines LNWL, MW, HNWL, HW30 and HW100 are available for the entire Austrian stretch of the Danube. Figure 51 provides an overview of relevant water level lines, clearly showing the location of power plants and locks, which are visible as abrupt vertical changes of the water level altitude.

When method 2 is applied, the altitude of current water levels is determined in proportion (as a percentage) to the pre-calculated reference water levels of the gauging station. Thereby, the current water level is always located between two bounding reference water level lines. The upper boundary line is defined as 1 (or 100%), and the lower boundary as 0 (or 0%).Within the interpolation process, the percentage of the current water level related to the boundary lines is calculated for the cross section. Subsequently, the difference of absolute altitudes between the two reference lines is multiplied by the percentage for the section gained from the interpolation process. The current water level in absolute altitudes for the selected cross section can be found by adding this value to the altitude of the lower boundary reference line.

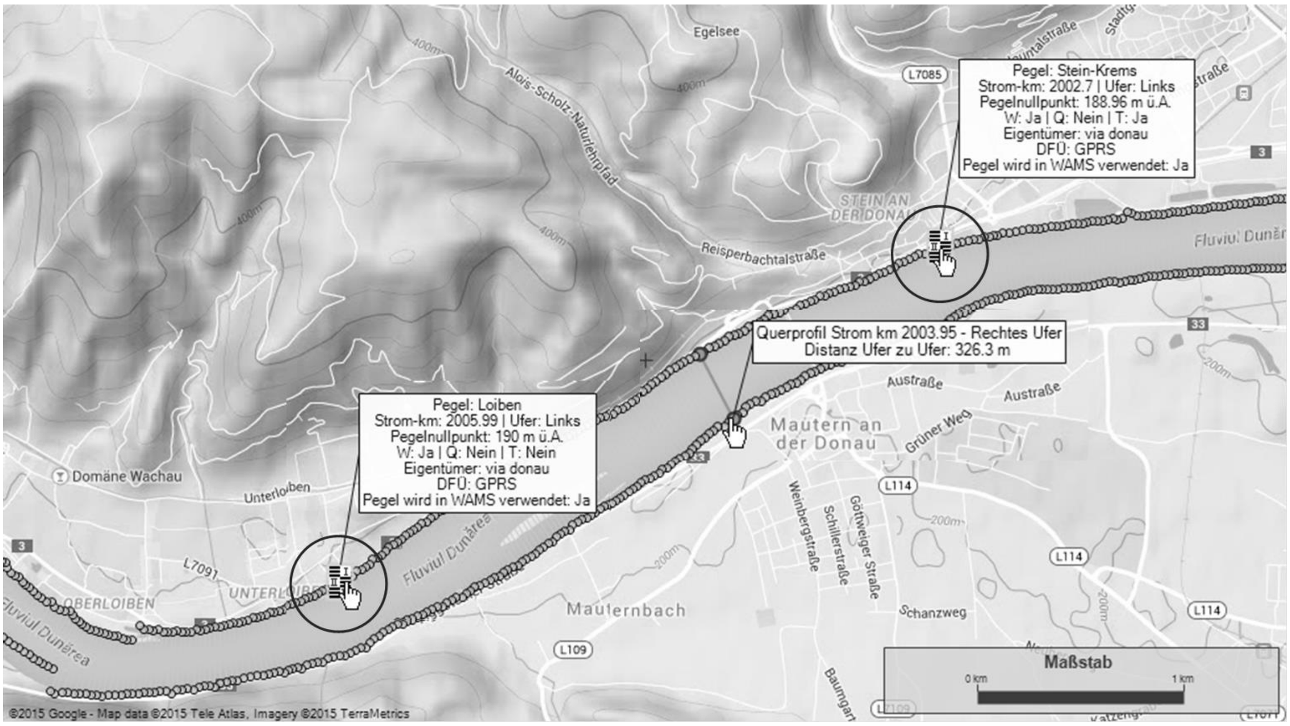


Figure 49: Example WAMS: water level calculation for a cross section at rkm 2003.95 using water level information of gauging station Loiben [2005.99] and gauging station Krems-Stein [rkm 2002.7]

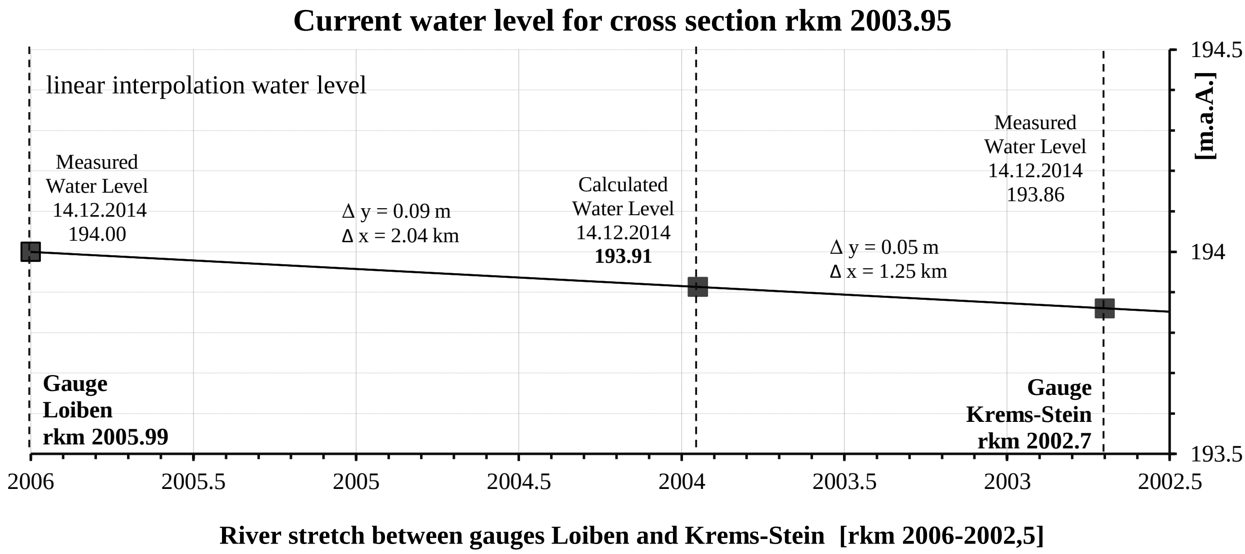


Figure 50: Calculation of current water level for a selected cross-sectional profile based on direct linear interpolation between gauging stations.

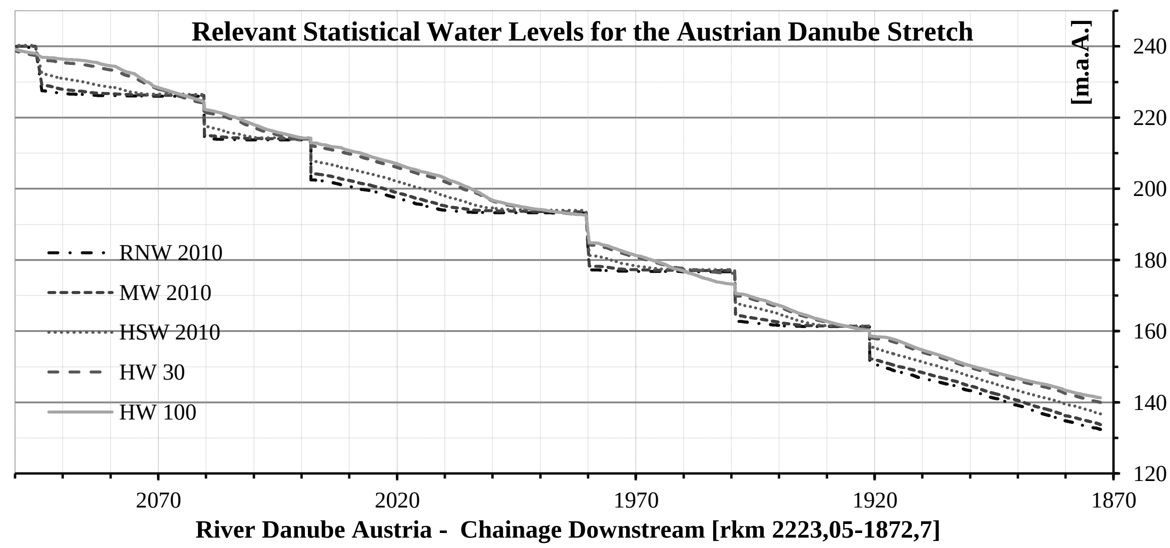
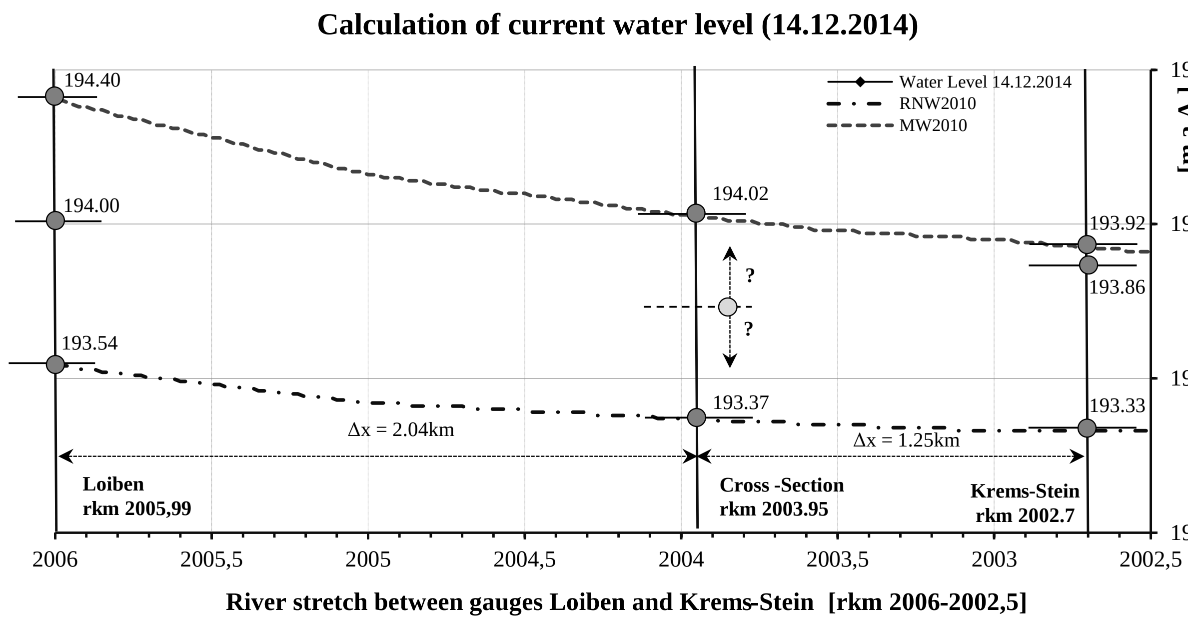


Figure 51: Visualization of pre-calculated water level lines based on a 2D-HN calculation (RNW2010, MW2010, HSW 2010, HW30, HW100) that are used in the Austrian WAMS in the case of a failure of important gauges [VIADONAU] [WAMS].



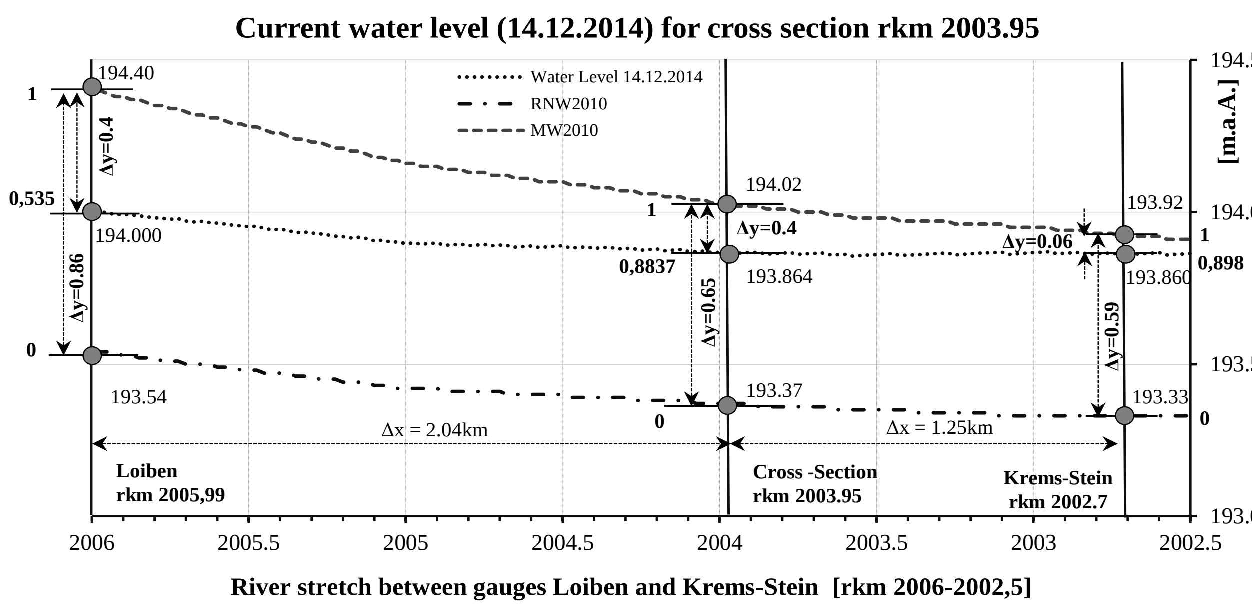


Figure 52: Calculation based on multiple linear interpolation, using pre-calculated relevant water level lines and gauging data

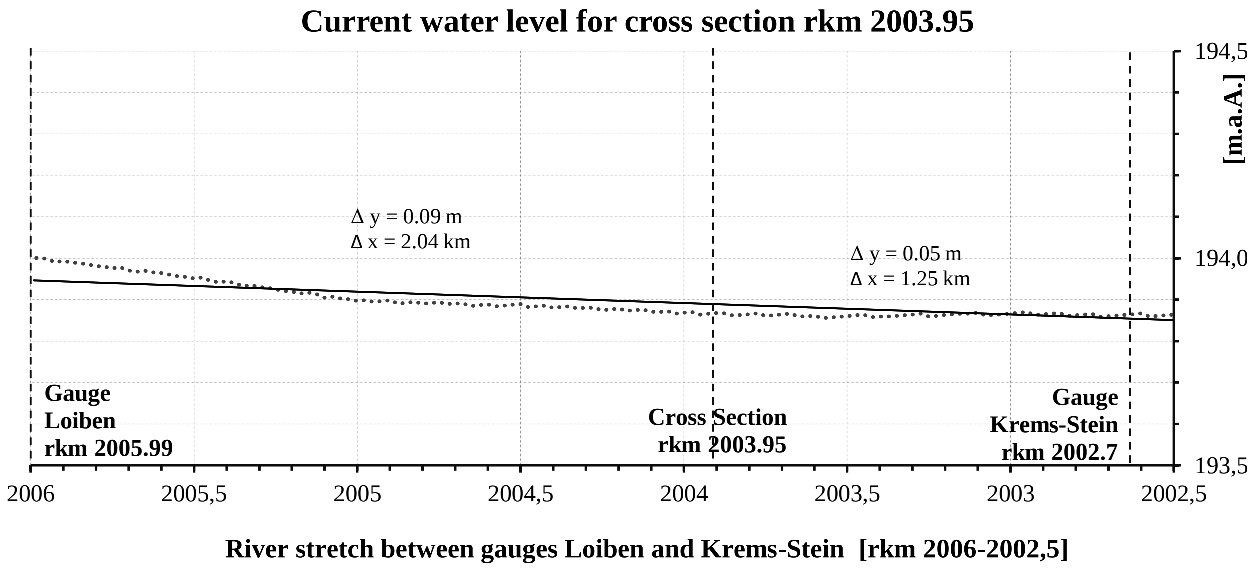


Figure 53: Deviations between direct linear interpolation and linear interpolation between water level lines for calculation of current water level of cross section rkm 2003.95.

The resulting current water levels are presented in Figure 51 for all cross sections between the gauging stations. The calculated water level based on applying this method amounts to 193.86 meters above the Adriatic Sea Level. For this cross section, the deviation between the results of the applied interpolation methods amounts to 0.05m. Figure 53 specified the deviations between the two interpolation methods along the length of the river section between the gauging stations. In backwater areas of power plant, the results of these methods show the lowest degree of deviations. With increasing level of discharge, deviations between the methods also increase. For extreme flow conditions, method 2 will provide more stable calculations and should therefore be preferred.

4.2.4 Example WAMS: gauges and information on water levels

In the Austrian waterway asset management system 82 gauging stations with automated data transmission via GSM, and mostly individual power supply, provide water level information for calculations and evaluations.

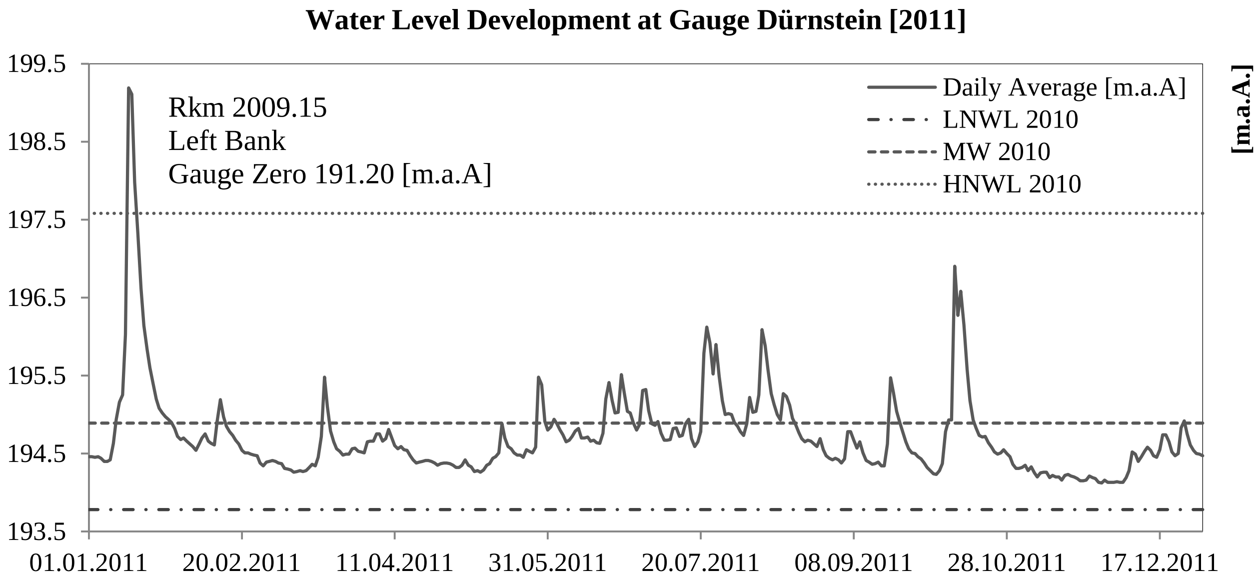


Figure 54: Visualization of water level development for 2011, also including statistical water levels as LNWL, MW and HNWL; WAMS- available gauging data include daily minimum value, daily average and daily maximum value

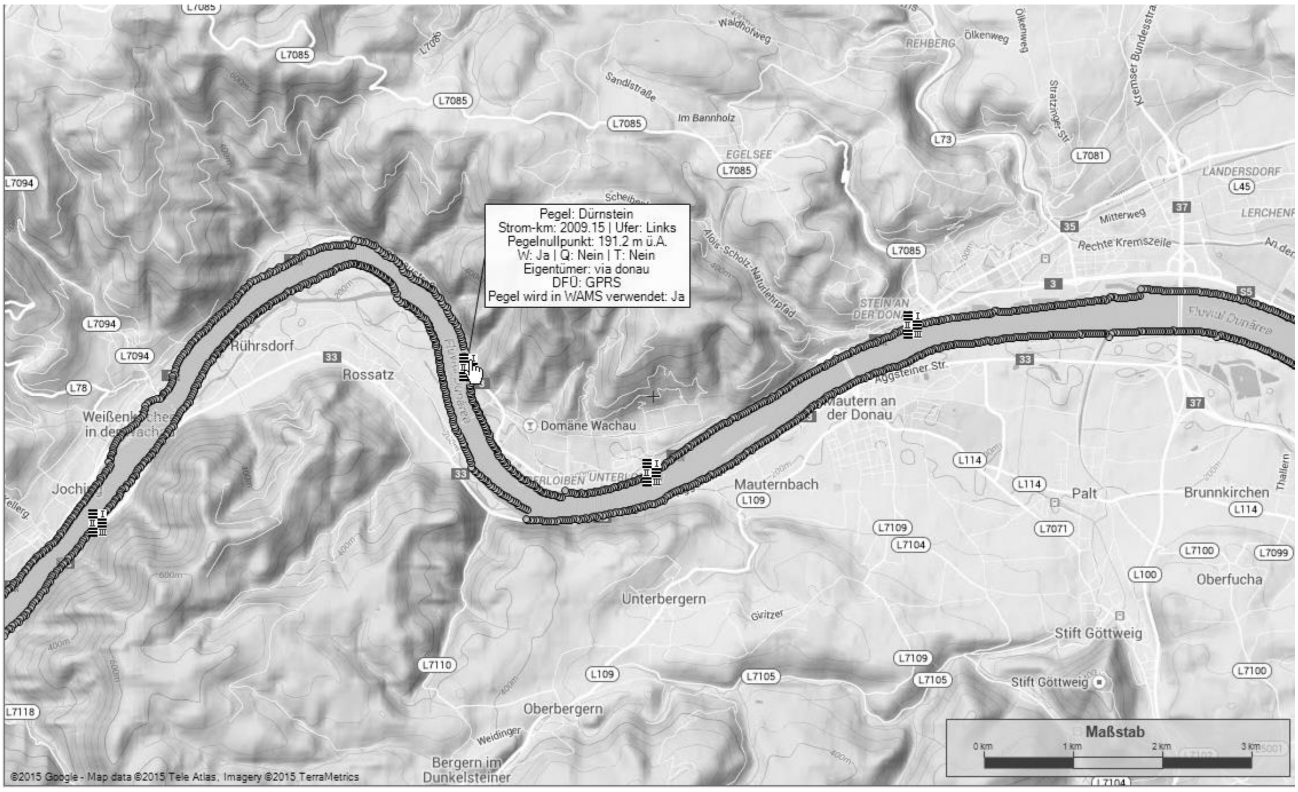


Figure 55: Visualization of water level gauges in the Austrian waterway asset management system

Figure 55 exemplifies the visualization of gauging stations provided by the WAMS software tool. In addition to the daily average, the minimum and maximum value of the water level can be displayed as well as relevant statistical water levels, as shown in Figure 54. An evaluation of water level information, for a period of one year results in the derivation of characteristic hydrographs of the water level as shown in Figure 54 gauge Dürnstein. Water levels represent an important system element for any waterway asset management system. Together with defined fairway parameters represented by the fairway channel as well as current riverbed surveys, information on current fairway conditions might be provided for the navigation industry for any day and any cross section.

Figure 56 exemplifies the visualization of cross section in the WAMS-software tool. Additionally, the WAMS software tool provides the feature of an animation of water level and riverbed development for each cross-sectional profile, including the availability of any pre-defined fairway path and any day of a year.

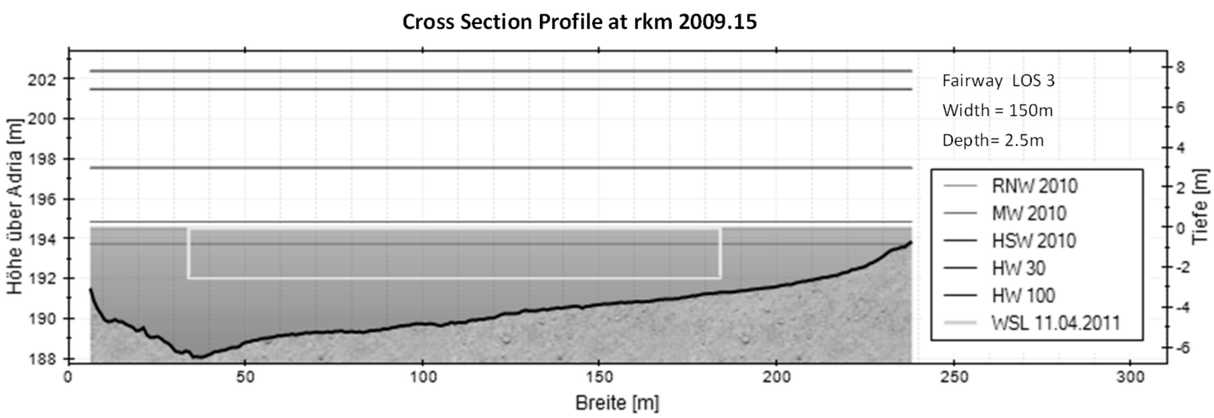


Figure 56: The illustration of cross section in WAMS includes current water levels together with the defined fairway and the latest survey of the riverbed as well as all important statistical water levels for this cross section.

4.2.5 System boundaries and restrictions for water level gauges

Reliable data from water level gauges is the basis for calculating a water level model and forms the backbone for assessing fairway availability together with data from bathymetric surveys. Optimum locations of automatic gauging stations depend, as mentioned afore, on the local hydro-morphological characteristics of the river with gauging stations at sections having the most significant changes in the hydraulics of the riverbed/critical sectors (Figure 57).

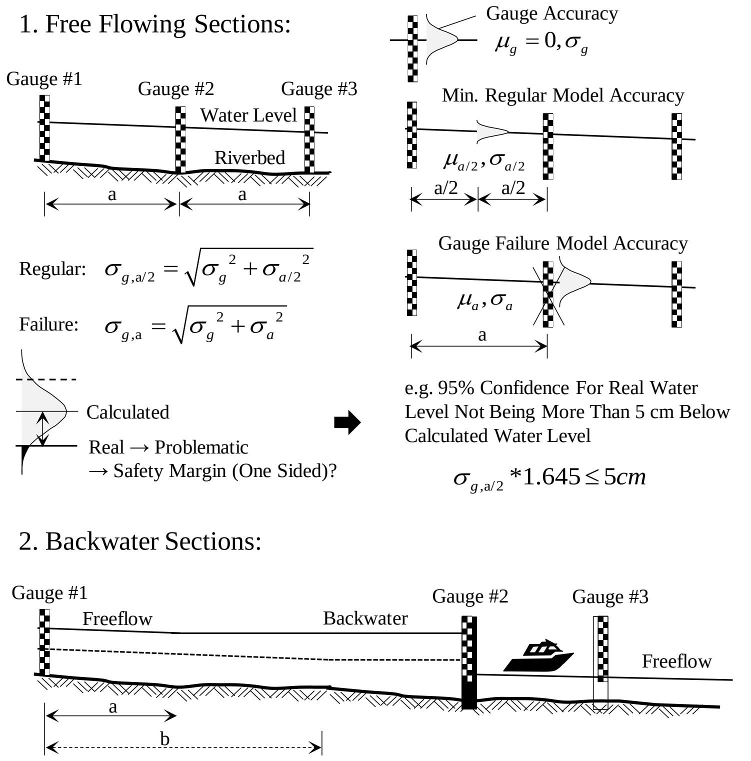


Figure 57: Situating water level gauges and possible accuracy considerations regarding calculated water levels

In order to acquire continuous information at least every hour, an automated transmission (e.g. via GSM) of measurement data as well as an independent power supply for all gauging stations are necessary [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

For a comprehensive waterway asset management system, every water level model that can be calculated or actualized and maintained for entire national river sections with an accuracy of ± 5 cm at 95% confidence level or less, should be sufficient. However, in order to allow an automatic processing water level gauges need to have a high reliability and the system should be able to filter errors and compensate for at least a possible failure of a neighboring gauge. Based on such a standard, almost real-time availability and water level information may be provided to customers. For calculating the sufficient number of gauges and gauge density, the average slope of the riverbed is an important factor along with the number of gauges increasing with slope [Hoffmann M. & Haselbauer K. & Blab R. 2014].

On the upper Danube from Kehlheim to Gönyü (rkm 1,791.33 – 2,414.72), the slope is 37 cm/km. On the central Danube down to Turnu Severin (rkm 931.00) this value is 8 cm/km and for the lower Danube down to the Black Sea it is only 4 cm/km.

For the implementation of a full WAMS on the entire Danube River, accurate and reliable information on water levels is essential. According to Figure 57, the total accuracy of calculated water level is a function of both accuracy and density of gauges as well as the water level model itself. It is recommended to set up a project with external experts for updating existing and/or calculating new 1D/2D water level models for the entire Danube River that can be incorporated in a WAMS [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

4.3 Riverbed model

4.3.1 Data requirements and goals

The model of the riverbed is the second important system element of any waterway asset management system. In order to allow riverbed modelling, regular surveys are required. For riverbed survey, different survey methods aregenerally available showing varying degrees of accuracy and different survey costs. Depending on the modelling target (basic inspections of available fairway depths or evaluation of sedimentation processes), the application of basic echo sounder surveys or areal multi-beam surveys is considered as appropriate.

The selection of a proper survey method is especially important for critical shallow sections in free-flowing river sections. Depending on the technical equipment, either absolute altitudes of the riverbed are determined by the survey, or the elevation of the riverbed is given relative to a reference water level line. Regardless of the survey method or survey purpose, raw data have to be examined within a post-processing regarding their plausibility and outliers. As the amount of data gathered by multi-beam surveys is mostly too big for subsequent calculations because common computing times are very time consuming, the data must be thinned out. For this purpose, grid generators form the basis for further model design. Furthermore, appropriate survey intervals are required for the characterization of riverbed developments and thus the development of fairway depths as well. As a result, it is evident, that the accuracy of provided information on fairway depths depends not only on current water level information, but also on the frequency of riverbed surveys. This also applies for the description of sedimentation; processes that are not measured must either be predicted with appropriate software-programs or cannot be represented. An empirical analysis of sedimentation erosion processes therefore requires a sufficient frequency of riverbed surveys that should be evaluated separately for each shallow section. The following section of this thesis provides an overview of common survey methods and gives an insight into the algorithms used for triangulation, creating contour lines and shading, as well as finally presenting the results of the riverbed model based on an example of shallow section development with the WAMS-software-tool.

4.3.2 Bathymetry and methods in riverbed surveying

Bathymetry[39] research deals with underwater depth of lakes, rivers or ocean floors. Bathymetric (or hydrographic) charts usually contain contour lines, also called depth contours or isobaths, and selected depths (soundings), and aim at providing depth information for inland navigation as an important influencing parameter for possible vessel draught and transport safety. Early techniques used pre-measured heavy rope or cable lowered over a vessels' side. These methods of depth measurement have to be considered as inaccurate and inefficient.

Today, the data that is necessary to draw bathymetric charts is usually provided by echo sounders (also referred to as sonar). Sonar[40] stands for SOund Navigation And Ranging, and is divided into two types of technology: passive sonar focuses on listening to the sound made by vessels; active sonar emits pulses of sounds and listens for the echo. Echo sounders[41] are used to determine the depth of water by transmitting sound pulses into water.

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[39] https://en.wikipedia.org/wiki/Bathymetry; *(Last access: 02.09.2015)*
[40] https://en.wikipedia.org/wiki/Sonar; *(Last access: 02.09.2015)*
[41] https://en.wikipedia.org/wiki/Echo_sounding; *(Last access: 02.09.2015)*

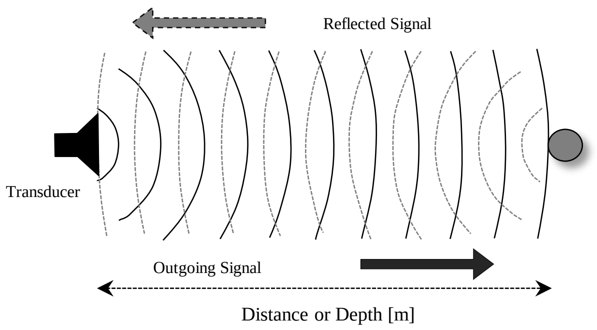

Figure 58: Basic principle of echo sounding with an outgoing signal given by a transducer is reflected by an object, for example the river bed.

Echo sounders generally comprise of a transmitter and a receiver. The transducer emits a sound pulse which is reflected by the riverbed and subsequently registered by the recipient (compare Figure 58). The time interval between emission and return of a pulse is recorded and used for calculations of water depth along with the speed of sound in water during this time. The water depth *(D_w)* is calculated according to equation (45) by multiplying half the time difference *(Δt)* from the signal's outgoing pulse, to its return by the velocity of sound in the water *(v_w)*. The velocity of sound in the water is measured with an acoustic velocity sensor for different layers of waters and is therefore considered as known [Kern, A. 2008]. A typical assumption for the average speed is approximately 1.5 kilometers per second. For precise applications of echo sounding, such as hydrography, the speed of sound must also be measured, typically by deploying a sound velocity probe into the water.

$$D_w = \frac{1}{2} \cdot v_w \cdot \Delta t \tag{45}$$

The velocity of sound in water varies with water temperature, density (which depends inter alia on the salinity of water) and the prevailing pressured. Therefore, the velocity of sound is different in any body of water and any layer of the water body. This is because the velocity of sound in water changes with changes of temperature and pressure over the depth of a water body such as a river. The selection of the frequency emitted by the transducer depends on the depth of the examined river or water body. For shallow water bodies (e.g. rivers), transmitters with a higher frequency are selected. For deep water bodies, low frequencies are preferred since they provide a higher accuracy as they are able to permeate water layers better through using a lower attenuation. Nevertheless, they offer a lower resolution [Kern, A. 2008].

Common echo sounding[42] equipment that is available on the market generally differs regarding vertical accuracy, resolution, acoustic beam-width of the transmit/receive beam and the acoustic frequency of the transducer. The majority of hydrographic echo sounders are dual frequency, meaning that a low frequency pulse (typically around 24 kHz) can be transmitted at the same time as a high frequency pulse (typically around 200 kHz). As the two frequencies are discrete, the two return signals do not typically interfere with each other. There are many advantages of dual frequency echo sounding, including the ability to identify a vegetation layer or a layer of soft mud on top of a layer of rock.

An accurate measurement of water depths requires a very precise positioning of the survey vessel. Furthermore, appropriate coordinate systems must be determined in advance. As all types of echo sounders operate on sound basis, only measurements of relative distances between transducer and riverbed can be provided. Thus, determinations of the position of the vessel as well as a precise alignment of the transducer are required for a comprehensive and absolute orientation. This geo-referencing is given by the means of GPS and motion sensors (Figure 59).

[42] https://en.wikipedia.org/wiki/Echo_sounding; *(Last access: 02.09.2015)*

In addition to sound velocity sensors, different types of GPS and GPS-compasses are also required for the three-dimensional localization of the vessel and the transducer. In order to increase the accuracy of positioning, two GPS-receivers are applied simultaneous. This method is termed as DGPS[43] (differential global positioning system). It provides an improved location accuracy of about 10 cm in the case of appropriate implementation. The first receiver, also referred to as base station, is located at a point with known coordinates for the duration of the entire survey. The second receiver (movable rover), however, is used for the determination of unknown points. In order to ensure the measurement of accurate vertical water depths, motion sensors for vessel movements are imperative. This motion sensor covers all movements of the vessel including roll-, pitch-, yaw and stroke movements that have to be taken into account for the determination of fairway depth. Both survey and post-processing are based on specialized software solutions such as for example QUINSy[44], for example. Figure 59 shows the calculation principle of absolute altitudes of the riverbed using RTK[45]-GPS. The determination of the water level is determined by combining current gauge level and height position of the GPS-system. The absolute altitude of the riverbed may subsequently be calculated by reducing the measured altitude with the offset between GPS-antenna and transducer, and the measured water depth.

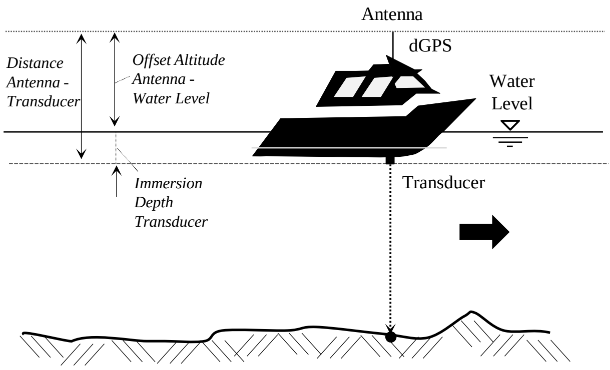

Figure 59: Calculation of absolute altitudes of the riverbed and water level using RTK-GPS.

4.3.2.1 Single-beam echo sounding

Single-beam echo sounding methods measure water depth based on one single vertically emitted acoustic signal (visualized in Figure 60). In order to obtain accurate depth information on the obtained riverbed, dual frequency echo sounders are commonly applied. Higher frequencies have the disadvantage that they are not only reflected by the riverbed, but also by suspended particles and mud layers. By contrast, lower frequencies permeate through mud layers and are only reflected by the riverbed. Data gathered by single-beam surveys show a lower density and therefore often require interpolation processes between known points of the riverbed. The data may be used for derivation of isobaths and navigational charts. The relatively small amount of data allows for fast processing and evaluation [Kern A. 2008].

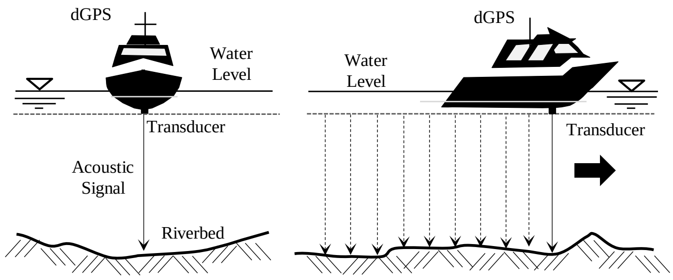

Figure 60: Principle of single-beam echo sounding.

Single-beam echo sounders allow a survey of water depth that is restricted to the linear paths of the riverbed that are covered with acoustic signals. Ahead in this thesis, the implementation of single-beam surveys, the survey path is defined as well as the grid and the cross-sectional or longitudinal profiles that are subject of the survey, as illustrated in Figure 61. Surveys of *longitudinal* profiles can be described as linear

[43] *https://en.wikipedia.org/wiki/Differential_GPS; (Last access: 02.09.2015)*
[44] *http://www.qps.nl/display/qinsy/main;jsessionid=03291DA9ECD0F2597A56515D5E42B932; (Last access: 02.09.2015)*
[45] *https://en.wikipedia.org/wiki/Real_Time_Kinematic; (Last access: 02.09.2015)*

measurements parallel to the fairway path. Thereby, a fast check of condition and changes of riverbed condition is possible. A result of longitudinal measurements with single-beam echo sounders is a low amount of data with a low accuracy. Surveys of cross-sectional profiles are described as linear measurements of cross-sectional profiles with defined distances between 25 to 100 meters, serving for regular surveys and allow for a comparison of systematic profile development over time [Hoffmann, M. et al. 2014d]

However, one disadvantage of single-beam echo sounding is that only punctual information of riverbed altitudes gathered along survey lines may be obtained, but large areas of the riverbed are not covered. Representative models of the riverbed are thus restricted regarding their significance. This sounding type is described as rather inaccurate and is therefore mainly used to control depth information or to supplement non-critical river sections. This also includes river stretches where a high flow velocity does not allow the application of multi-beam [Mic, L.-M. 2013].In practice single-beam echo sounding is also used for surveys of shallow sections and preparation of measures. Single-beam survey data are imported into the WAMS-software by default and used for the creation of isolines and depth maps.

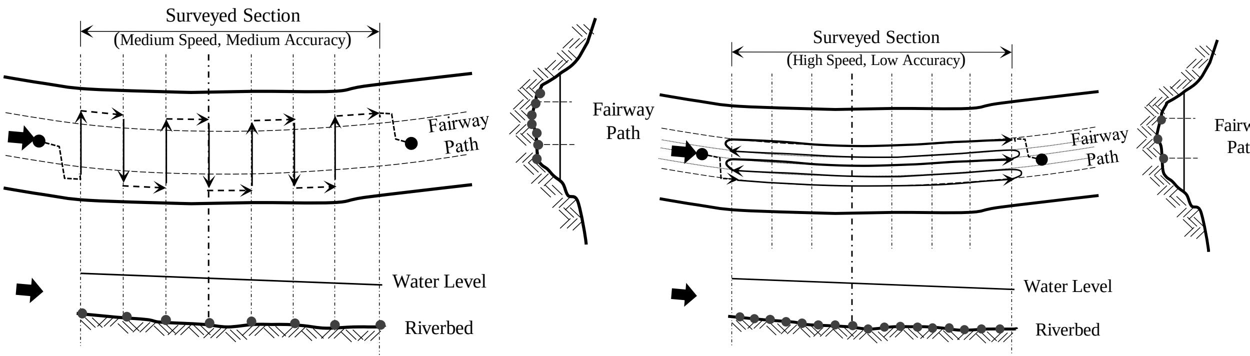

Figure 61: Riverbed survey approaches using single-beam survey equipment: survey of cross sectional profiles (regular surveys) and longitudinal profiles (fast check, marking) [Hoffmann M., Haselbauer K., Blab R. 2014]

4.3.2.2 Multi-beam echo sounding

Today, a common method of riverbed surveying is the application of multi-beam echo sounders also referred to as MBES[46], which use hundreds of very narrow adjacent signals arranged in a fan-like swath of typically 90 to 170 degrees across. The transducer simultaneously emits several acoustic signals in a fan shape manner. The tightly packed array of narrow individual beams provides very high angular resolution and accuracy. In general, a wide swath, which is depth-dependent, allows a vessel to map a bigger area of the riverbed in less time than single-beam echo sounders by making fewer passes.

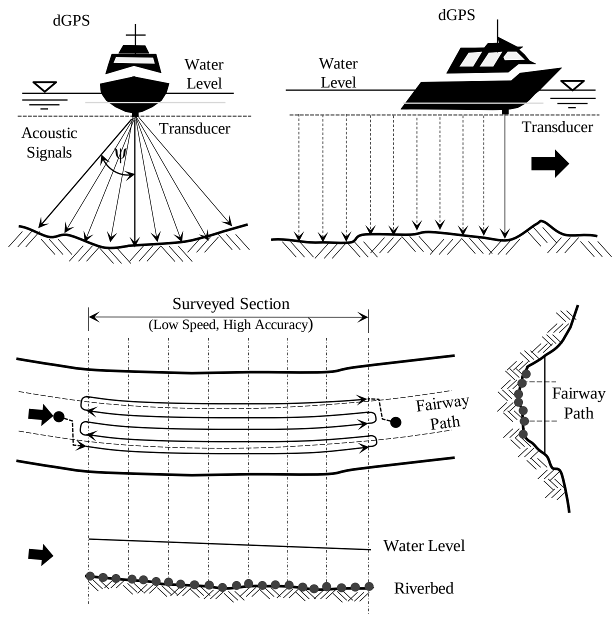

Figure 62: Principle of multi-beam echo sounding and survey principle with multi-beam based on overlapping stripes

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[46] *https://en.wikipedia.org/wiki/Bathymetry; (Last access: 02.09.2015)*

For this survey method, the entire riverbed is surveyed within a relatively short time period, depending on factors such as the opening angle of the transducers and the current water depth of the river as well as the width of the fairway. The opening angle of the transducer may be reduced or enlarged as required [Mic, L.-M. 2013]. The water depth D_w for each acoustic signal can be calculated according to equation (46) with the emission angle ψ being visualized in Figure 62.

$$D_w = \frac{1}{2} \cdot v_w \cdot \Delta t \cdot \cos(\psi) \tag{46}$$

One advantage of multi-beam echo sounders, compared JIB-multi-beam sounders (see chapter 4.3.2.3), is that they allow surveys of river sections with a water depth of approximately 2.80 m (average depth of the Danube) where they can measure a 8 to 10 meter wide stripe (assuming an opining angle of 130°) of the riverbed, whereas common JIB sounders are only able to measure a 2.8 meter wide stripe (for 8 transducers with an opening angle of 9°). The width of the measured stripe has to be reduced since the edge beams cannot be used for the determination of water depth due to a declining accuracy [Kern A. 2008]. The survey of a shallow section is carried out by measuring overlapping stripes of the fairway path, according to Figure 62. The number of survey paths depends on the width of the river and the fairway as well as on current water depths. The Austrian waterway asset management system allows the import of multi-beam data related to any reference level. Multi-beam surveys are applied when detailed information of shallow section is necessary, as in riverbed measurements, before and after dredging measures and the cubature of a dredging measure has to be calculated. Multi-beam surveys allow a systematic comparison of the riverbed development over time and also the calculation and visualization of sedimentation and erosion processes. Multi-beam echo sounding may provide a higher data density allowing for more accurate evaluations. A big disadvantage of this survey method is that substantial financial efforts are required.

4.3.2.3 JIB-Multi-beam echo sounding

JIB-multi-beam echo sounding systems consist of several individual transducers which are mounted on a jib below the vessel. The number of transducers, and their horizontal distance from each other, depend on the density of cross sectional profiles, average water depth (shallow or deep water bodies) and the opening angle of the transducer. Individual transducers measure simultaneously, but usually send at different frequencies in order not to interfere with each other. A number of factors, such as water depth, required density of profiles, and the opening angle of the related transducer, influence the applied frequencies.

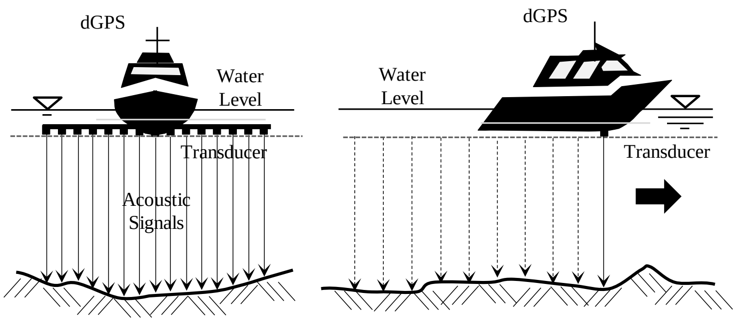


Figure 63: Principle of bottom and side scan sonars

Similar to single-beam surveys, cross-sectional or longitudinal profiles are measured based on a given grid, as presented in Figure 61. In this case, an appropriate grid is required allowing for comprehensive areal riverbed surveys while covering the shortest possible survey path at the same time [Kern, A. 2008]. Theoretically, this type of sounding system would be able to scan the entire riverbed, but this process must be considered as very time-consuming. This survey method is rarely applied but is used both for surveys in shallow and depth water bodies [Mic, L.-M. 2013].

4.3.3 Methods in grid generation, contouring and visualization of 3D-Data

Due to its density, a 3D-cloud of raw data of riverbed surveys is barely readable and not suitable for illustrations as navigational charts or models of the riverbed. Therefore, measured riverbed data, provided in Web Mercator – projection, is further processed and converted into two or three-dimensional grids and is used for the creation of contour lines and shaded charts of the riverbed. In order to map changes of riverbed altitude between several consecutive surveys, each measured riverbed point must furthermore refer to a constant reference point. In order to provide features as cubature calculations or difference maps, it is important to create an appropriate database structure for waterway asset management systems.

A digital topography, such as the riverbed, can be represented in either vector or raster format. Vector format uses a series of irregularly spaced elevation points connected by lines into a triangulated irregular network (TIN). Raster format divides the topographic surface into equally spaced intervals or a gridded array, and then displays the elevation value for each grid cell (called a digital elevation model or secondary DEM)[47].

In order to be as flexible as possible in terms of analysis options and applications, the riverbed model of the Austrian WAMS offers both the integration of TIN-data sets and raster-DEM. In the case of secondary elevation models (raster-DEM), a uniform grid is placed over the given terrain (e.g. riverbed). Equally spaced points in the center of each square grid cell represent the elevation of terrain. Elevations between the reference points of the grid can be calculated by bilinear interpolation methods. In order to reproduce the surface as accurately as possible, the mesh size must be defined sufficiently tight, so that important characteristics of the terrain can be mapped. Typical mesh sizes of local or regional models vary from 2 up to 500 meters. For the riverbed model of the Austrian waterway asset management system, the mesh size is determined with 5 meters[48] but can be set for any value from 1 cm to 100 m with huge differences in calculation time.

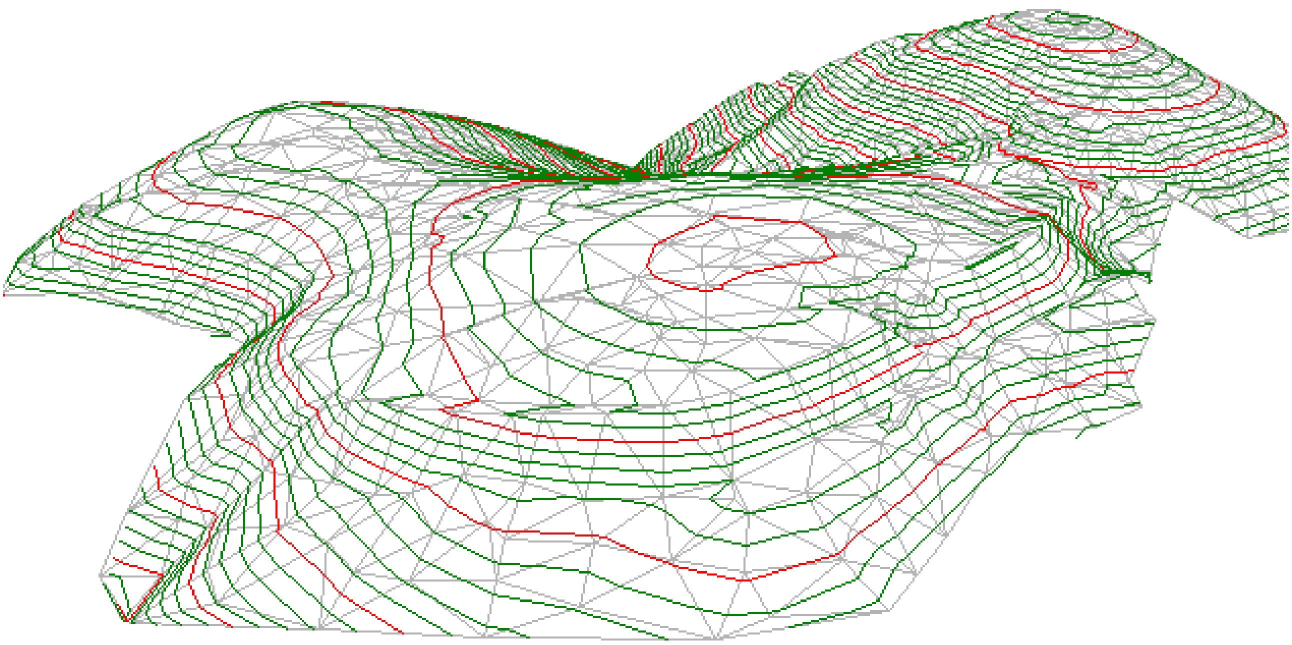
When vector-based triangulated irregular networks (TIN[49]) are applied for riverbed modelling, the surface of the riverbed is mapped based on a 3D-point cloud. Irregularly distributed nodes and lines with three-dimensional coordinates (x, y, and z), which are arranged in a network of non- overlapping triangles, thereby represent the physical riverbed. Three-dimensional visualizations are readily created by rendering of the triangular facets. In regions where there is little variation in surface elevation, the points may be widely spaced, whereas the point density is increased in areas of more intense variation in elevation. TINs are typically based on DELAUNAY triangulation[50]. With the method of DELAUNEY triangulation, points are linked to triangles in compliance with the boundary condition that no further additional points are included in that circuit, which comprises the triangular points. Regarding mapping and analysis, the application of TINs has a number of advantages compared to raster-DEM, because points of a TIN are distributed variably based on an algorithm that determines which points are most necessary for an accurate presentation of the terrain. Therefore, TINs are used for calculations of all cubature needed for the purposes of waterway asset management. Data input is thus flexible and fewer data points need to be stored compared to raster-DEMs with regularly distributed points. Raster-DEMs are easier to handle regarding the analysis of surface slope.

[47] *http://serc.carleton.edu/vignettes/collection/42681.html; (Last access: 15.09.2015)*
[48] *https://de.wikipedia.org/wiki/Digitales_H%C3%B6henmodell; (Last access: 15.09.2015)*
[49] *https://en.wikipedia.org/wiki/Triangulated_irregular_network; (Last access: 15.09.2015)*
[50] *https://en.wikipedia.org/wiki/Delaunay_triangulation; (Last access: 15.09.2015)*

Through the regular grid, raster-DEMs are preferred in terms of comparison of survey data and changes of the riverbed.

Nautical charts for inland navigation are substantially based on the visualization of depth information based on isobaths. Isobaths are contour lines connection points of equal depth (underwater depth). An elevation contour dataset (isobaths) represents lines of equal elevation above a given levels such as Adriatic Sea level. Nevertheless any other reference level may be applied. Contour levels of inland waterways are commonly related to the representative statistical water level line LNWL. From these contours, a sense of the general terrain can be determined. With increasing density of contour levels, elevations of the riverbed can be mapped more exact. When isobaths are close together the magnitude of the gradient is large. For nautical charts of inland waterways a vertical distance (contour interval) of 0.5 meters is considered as sufficient. In addition, the information regarding navigability is supplemented with punctual depth information. Elevation contours are fast to render, and are supported by a wider variety of software applications. Contouring in the Austrian waterway asset management systems is based on the CONREC[51] algorithm, which will be described in excerpts from the next chapter.

In order to increase the readability of navigational charts, contour lines are often combined with TINs. Figure 64 provides an example of a digital terrain model based on contour lines and triangulated irregular networks. Another possibility to enhance readability is coloring the areas between two adjacent contour lines. For this purpose, international harmonized color scales (color codes) are used for the visualization. In the presented Austria waterway asset management systems, eleven depth levels are used. The colored areas on these charts determine zones with water depth values varying between both bounding contour levels. This process is also known as shading and furthermore requires information about the order of contour levels. Shading of these areas is only possible when the polygons of the contour lines are closed. In some cases including plateaus or lowest points of a terrain, this process may be implemented easily. However, in order to allow coloring of areas between contour lines reaching peripheral areas of the survey, it is necessary to close these isobaths with a convex bounding polygon. The results of this modeling process in WAMS are presented in chapter 4.3.5.



[51] **Figure 64: Civilgeo - Example TIN-Surface with overlaid contour lines; http://support.civilgeo.com/knowledge-base/esri-tin-to-elevation-contour-shapefile/; (Last access 15.09.2015)**

4.3.4 Applied methods for grid generation, contouring and visualization of 3D-Data

4.3.4.1 Integration and contouring of raster-data

When raster data are used, complex thematic evaluations may be realized as flexible and particularly elegant. Raster-data consume more memory, but have the advantage of a simpler databased structure and do not require complex calculations of data links. In order to model the riverbed based on raster-data, the survey (river or shallow section) is divided into small, regular square cells with a 7.5 x 7.5 m grid based on Web Mercator projection. Since the projection in Web Mercator is not isometric, the grid used for the riverbed in WAMS has a mesh size of about 5 x 5 meters (Figure 65). Raster datasets that are imported into the WAMS contain information on position, orientation, grid size and type of value coding. Thus, not only corner coordinates are stored for each cell, but also the respective altitude of the related survey.

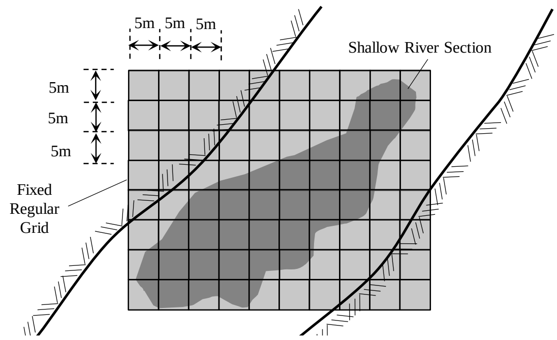


Figure 65: Basic grid for riverbed modelling with about 5 x 5 meter grid size

As a first step for the visualization of raster-data in the waterway asset management system, a definition of contour levels (elevation levels) is carried out. The reference level for contour lines might either be the current water level, a statistical water level as LNWL, or a reference sea level (contour lines of absolute altitudes). Thereafter, the contouring subroutine CONREC is applied for each contour level (height level). The data is thereby stored in a two dimensional array. This rectangular or square grid presents 4 points at a time. The center of each square is assigned a value corresponding to the average values of each of the four vertices. Each rectangle is subsequently divided into four triangular regions by cutting along the diagonals. Each of these triangular planes may be bisected by a horizontal contour plane. The intersection of these two planes is a straight line segment, and is part of the contour curve at that contour height [Bourke, P. 1987]. Figure 67 illustrates the intersection of the contour level and the 4 triangular planes as well as the resulting contour segments. As a result, the starting and stopping coordinates of the line segment are identified. Each square cell is treated this way. In the next step, the line segments belonging to one contour level are connected to a polygon. This polygon is also termed as contour line or isobath. This process is repeated for all contour levels. Figure 66 gives an example of creating a contour line of line segments which are obtained from the intersection of a contour level and the triangular planes obtained from the raster data.

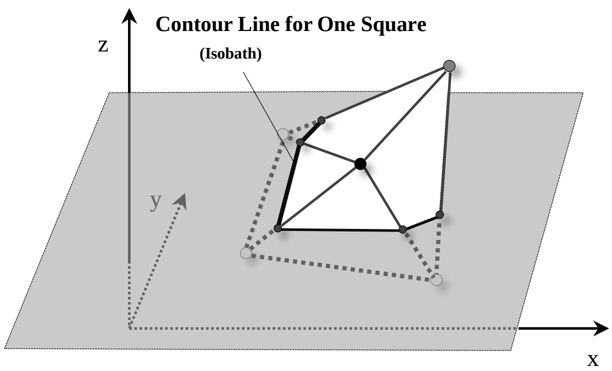


Figure 67: Contouring subroutine for visualizing three dimensional surfaces as isobaths for a two dimensional medium as navigational charts regular rectangles or squares on visualization based on [Bourke, P. 1987]

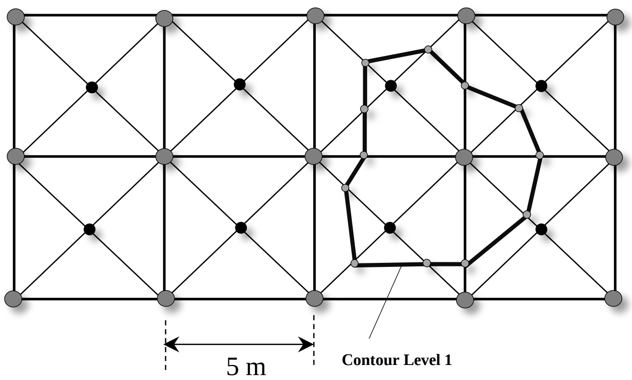


Figure 66: Example - Contour line for one contour level based on a grid of regular squares of 5 x 5 meters.

4.3.4.2 Integration and contouring of TIN-data

The Austrian waterway asset management system is capable of importing any number of triangles, each represented by four points (the starting point of a triangle is equivalent to the end point) with coordinates in Web Mercator projection. In addition to the triangles, a bounding polygon must be delivered in order to accelerate the further processing. The heights of the triangle points refer to the statistical reference water level LNWL.

In order to create contour lines based on TIN data, the vertices of the triangles are converted back to a regular square grid as presented in Figure 68. As a first step, all triangles are associated with a grid cell. Based on the method of inverse distance weighting, all vertices of the triangles are now used to determine the altitude of the center of the square cell.

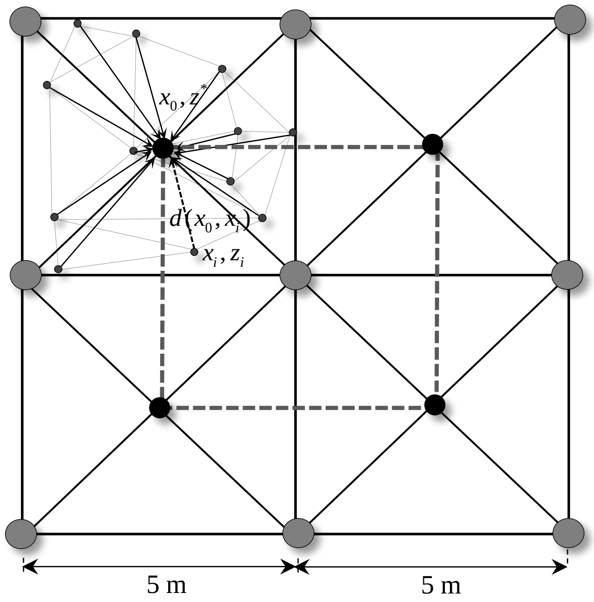


Figure 68: Converting TIN-data to a regular grid

Inverse Distance Weighting[52] (IDW) is a type of deterministic method for multivariate interpolation with a known scattered set of points, and is used for ple interpolation of spatial dependences of georeferenced data. The underlying assumption for this interpolation method is that punctually measured spatial data show certain similarities in values, depending on the distance in space. Thereby, the basic assumption applies, that the similarity of an unknown value to the known measured value decreases with increasing distance. Consequently, this means that the altitude-data are therefore less similar, the further apart they are. This basic assumption is considered when the method of inverse distance weighting is applied due to the fact that the measured altitude-value $z(x_i)$ is multiplied by a weight that is proportional to the inverse of the distance $d(x_0,x_i)$ between the estimated point x_0 (center of the square cell) and the measured point x_i (vertex of a triangle) using equation (47). In order to adequately account for the decrease of similarity, the distance is added by a power. The power has to be defined and should reflect the data situation (linear for linear surfaces; power of 2 or 3 for curved surfaces). A power of 2 is estimated for the distance weighting process for TIN-data in WAMS. In order to obtain the altitude of the center, this procedure is repeated for all vertices of the triangles within a grid cell.

$$z^*(x_0) = \frac{\sum_{i=1}^{n} \frac{z(x_i)}{d^2(x_0, x_i)}}{\sum_{i=1}^{n} \frac{1}{d^2(x_0, x_i)}} \tag{47}$$

By the means of IDW, the altitudes of the center of each cell are determined based on the vertices of the respective triangles within the grid. For the next steps the midpoints of square cells are connected to form a new square mesh. The derivation of the contour lines may then be carried out, following the same procedure as for raster data using the CONREC algorithm. Finally, a shading routine is applied to color the areas between contour lines as described afore in section 4.3.3.

[52] https://en.wikipedia.org/wiki/Inverse_distance_weighting; *(Last access: 15.09.2015)*

4.3.4.3 Difference map & development of sedimentation and erosion processes

Generally, contour lines, also referred to as isobaths of the riverbed, may be drawn and shaded as a chart for any riverbed survey. Since riverbed surveys take place on a periodical basis, a number of riverbed surveys exist for each shallow section that will, of course, be used for evaluations such as the determination of sedimentation and erosion processes. Due to the fixed cell size of the square grid, consecutive riverbed surveys can be superimposed and changes between two riverbed surveys may be determined as difference values, as presented in Figure 69. Thus, difference maps, of the riverbed between two surveys may be created that allow interpreting the characteristics of the development of shallow sections. In order to create these difference maps, the height values at the vertices of the grid of two surveys are subtracted, and the resulting difference value is considered as the new height value at the vertices of the square grid. By using the CONREC algorithm, the grid is again divided into four triangles that are subsequently intersected with the newly defined contour levels for the visualization of elevation differences. The areas between the resulting new contour lines are, in turn, colored based on the shading subroutine. These contour lines then connect points with equal sedimentation or erosion levels. As a result, the difference maps can be used for analysis purposes. TIN-data are converted into the regular square grid and the difference formation is carried out in the same manner as for raster data.

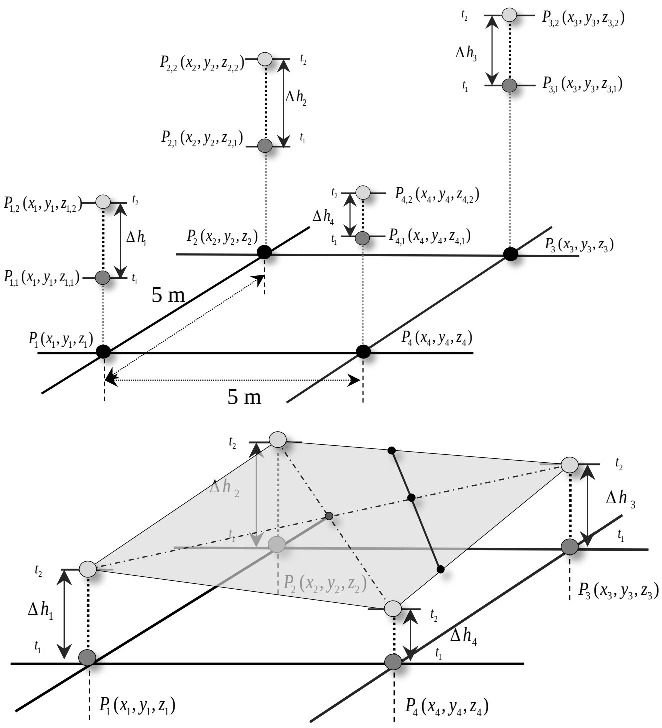


Figure 69: Overlaying raster data of the riverbed for surveys applied on different dates (t_1, t_2), with a calculation of altitude difference based on a square grid with a fixed cell size

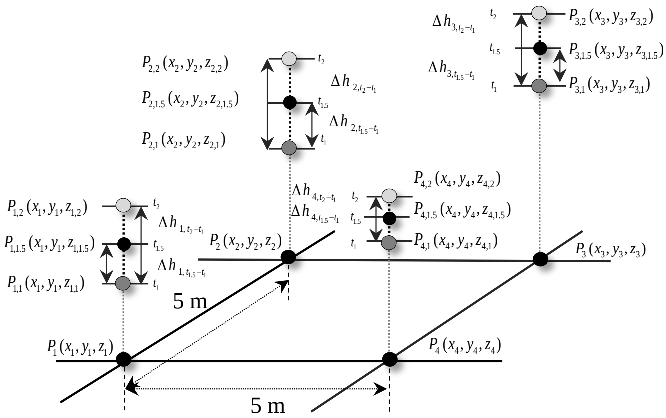


Figure 70: Interpolation of riverbed altitudes at grid vertices

Furthermore, it is also possible to implement a linear interpolation over time between height values at the vertices of the grid, allowing a representation of the riverbed development on a weekly or monthly basis, according to equation (48) (see Figure 70).

$$z_{2,1.5} = z_{2,1} + (z_{2,2} - z_{2,1}) \cdot \frac{t_{1.5} - t_1}{t_2 - t_1} \qquad \Delta h_{2,1.5} = z_{2,1.5} - z_{2,1} \qquad (48)$$

Each interpolation between two surveys of dynamic riverbed data at different points in time is, of course, an estimation and therefore subject to uncertainties. In addition to the time-component, it is also possible to integrate influence parameters, such as the prevailing discharge, into the interpolation algorithm and thus obtain a more expressive visualization of the development of the riverbed. Only at measurement occasions is an accurate representation of the actual riverbed possible. Abrupt changes of the riverbed exactly on the day of the next riverbed survey are very unlikely. Users of waterway asset management systems are free to choose the best variant of representation for their purposes.

4.3.5 Example WAMS: shallow sections

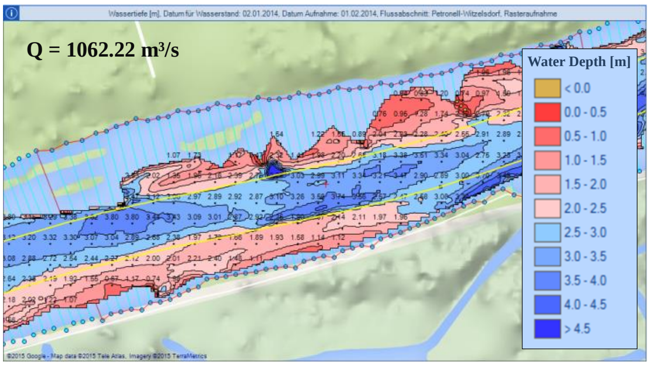
The following section will highlight the results of the modeling process of raster and vector data (TIN) of the riverbed, and contains contouring and shading subroutines of the WAMS-software as basic core process describing the condition development of the fairway. The described methods of calculation of altitude differences and interpolation between riverbed surveys are illustrated by examples provided by the WAMS-software. Riverbed surveys are of crucial importance, especially for critical shallow sections such as fords and lateral sedimentations. The Austrian Danube stretch, with a length of 350 rkm contains around 35 areas identified as critical for navigation, illustrated in Figure 71 . As an example for the illustration of maps of the riverbed, such as maps of fairway depths and difference maps, the shallow section ford Petronell-Witzeldorf is selected. This shallow section is chosen as an example since a particularly high number of riverbed surveys were implemented in recent years and highly dynamic sedimentation processes are in progress that very impressively demonstrate changes of the riverbed and, thus, the condition development of the fairway-infrastructure. This river section is also very special as it contains groin fields which were subject to constructive changes in recent years. This framework condition will not be subject to the following description of WAMS functionalities.

Figure 72 and Figure 73 demonstrate the results of raster data modeling in WAMS. The figures contain raster data from two multi-beam surveys implemented on 01.02.2014 and on 01.09.2014, within regular periodic measuring campaigns. The data are related to current daily water levels provided by the water level model and are converted into isobaths using the CONREC algorithm and finally colored using the shading subroutine. In the months between the two surveys, the altitudes of the riverbed are interpolated linearly over time according to equation (48). Figure 72 and Figure 73 represent the development of actual water depths during the year 2014. If the dredging measure which began in December 2014 is excluded, no further direct interventions regarding the riverbed development were applied, indicating the sedimentation was therefore subject to purely natural influences. The visualization of fairway depths in December 2014 clearly illustrates that actual available fairway depths were below 2.5 meters in many areas of the central fairway and thus constituted an obstacle to inland navigation. Figure 74 provides a difference map where altitude differences of raster data are calculated based on equation (48). The contouring and shading subroutines are applied as a standard process for the illustration of the charts, as described afore. The difference map in Figure 74 clearly visualizes sedimentations in the central fairway area, with partial landings showing an elevation exceeding one meter.

Figure 71: The figure illustrates the Austrian Danube Stretch with a length of 350 rkm; shallow sections and port areas marked in red; thereby the two free-flowing river section (Wachau & East of Vienna) are clearly visible; the location of ford Petronell-Witzelsdorf (part of the free-flowing section East of Vienna) is determined in the illustration at rkm 1893,2-1891,9.

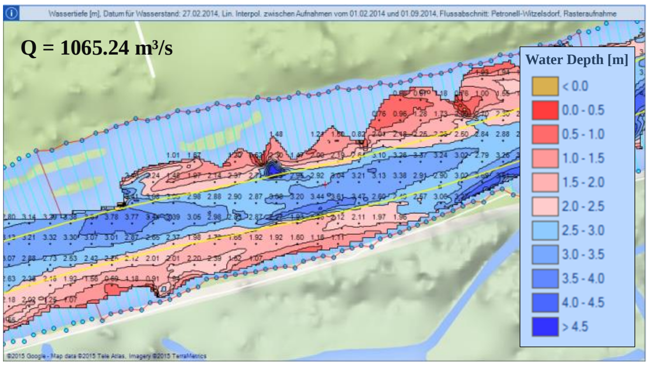
Ford Petronell-Witzelsdorf – January 2014

Water Level 02.01.2014; Survey Riverbed: 01.02.2014



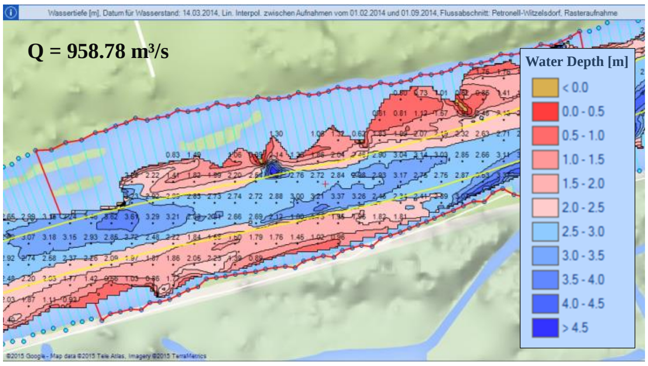
Ford Petronell-Witzelsdorf - February 2014

Water Level 27.02.2014; Interpolation Riverbed: 01.02. - 01.9.2014



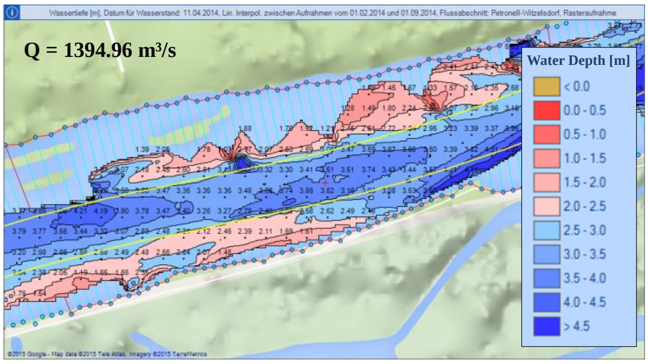
Ford Petronell-Witzelsdorf - March 2014

Water Level 14.03.2014; Interpolation Riverbed: 01.02. - 01.9.2014



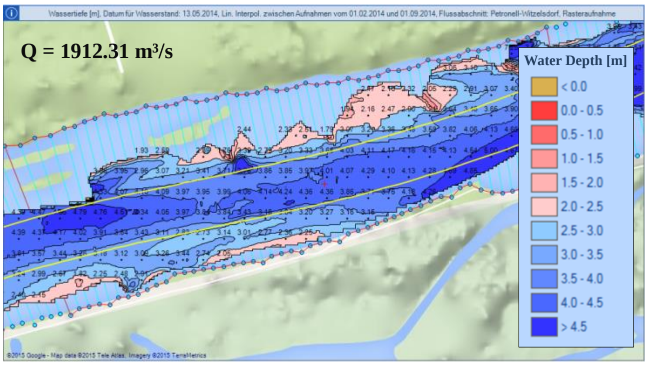
Ford Petronell-Witzelsdorf - April 2014

Water Level 11.04.2014; Interpolation Riverbed: 01.02. - 01.9.2014



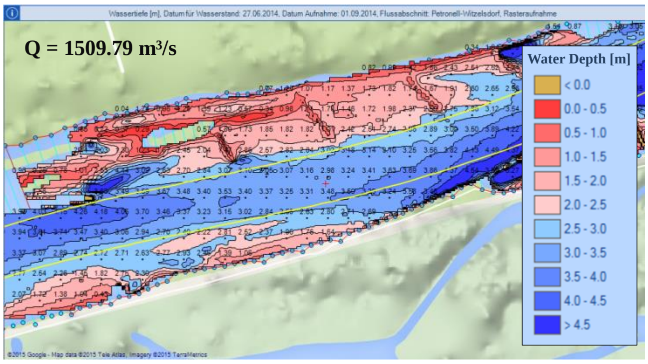
Ford Petronell-Witzelsdorf - May 2014

Water Level 13.05.2014; Interpolation Riverbed: 01.02. - 01.9.2014



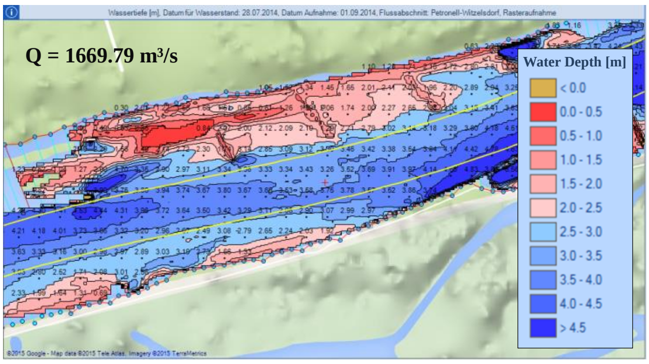
Ford Petronell-Witzelsdorf - June 2014

Water Level 27.06.2014; Survey Riverbed: 01.9.2014



Ford Petronell-Witzelsdorf - July 2014

Water Level 28.07.2014; Survey Riverbed: 01.9.2014



Ford Petronell-Witzelsdorf - August 2014

Water Level 15.08.2014; Survey Riverbed: 01.9.2014

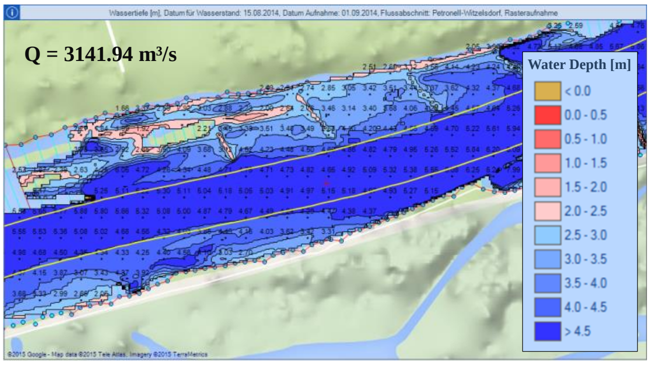
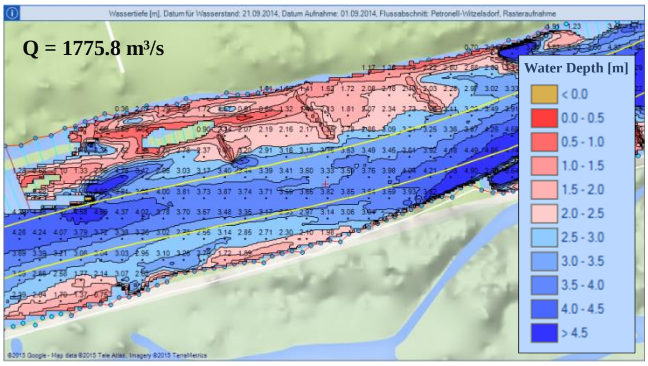


Figure 72: Development of water depths at shallow section – ford Petronell-Witzelsdorf – rkm 1891.9-1892.8 (January-August 2014) based on linear interpolation of raster data from multi-beam riverbed surveys carried out on 01.02.2014 and 01.09.2014 and current water levels (related discharge values were measured at gauge Wildungsmauer).

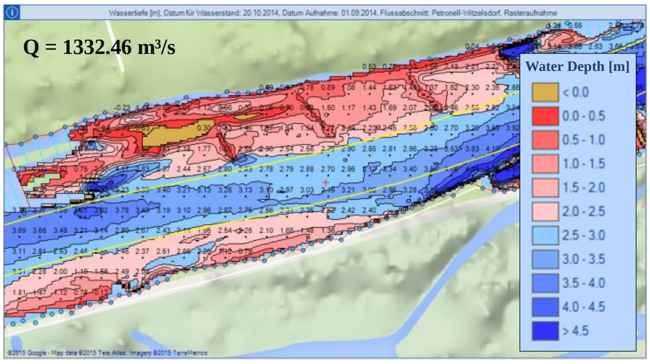
Ford Petronell-Witzelsdorf - September 2014

Water Level 21.09.2014; Survey Riverbed: 01.9.2014



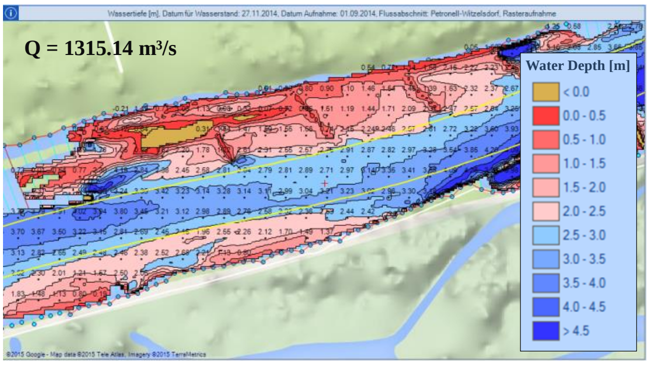
Ford Petronell-Witzelsdorf - October 2014

Water Level 20.10.2014; Survey Riverbed: 01.9.2014



Ford Petronell-Witzelsdorf - November 2014

Water Level 27.11.2014; Survey Riverbed: 01.9.2014



Ford Petronell-Witzelsdorf - December 2014

Water Level 15.12.2014; Survey Riverbed: 01.9.2014

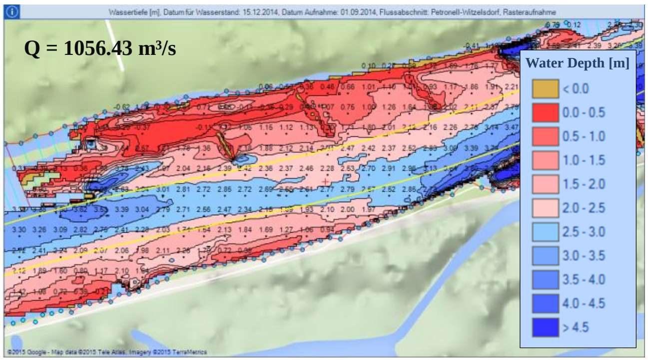


Figure 73: WAMS example - development of water depths at shallow section ford Petronell-Witzelsdorf – rkm 1891.9-1892.8 (September-December 2014) based on linear interpolation of raster data from multi-beam riverbed surveys carried out on 01.02.2014 and 01.09.2014 and current water levels (related discharge values were measured at gauge Wildungsmauer).

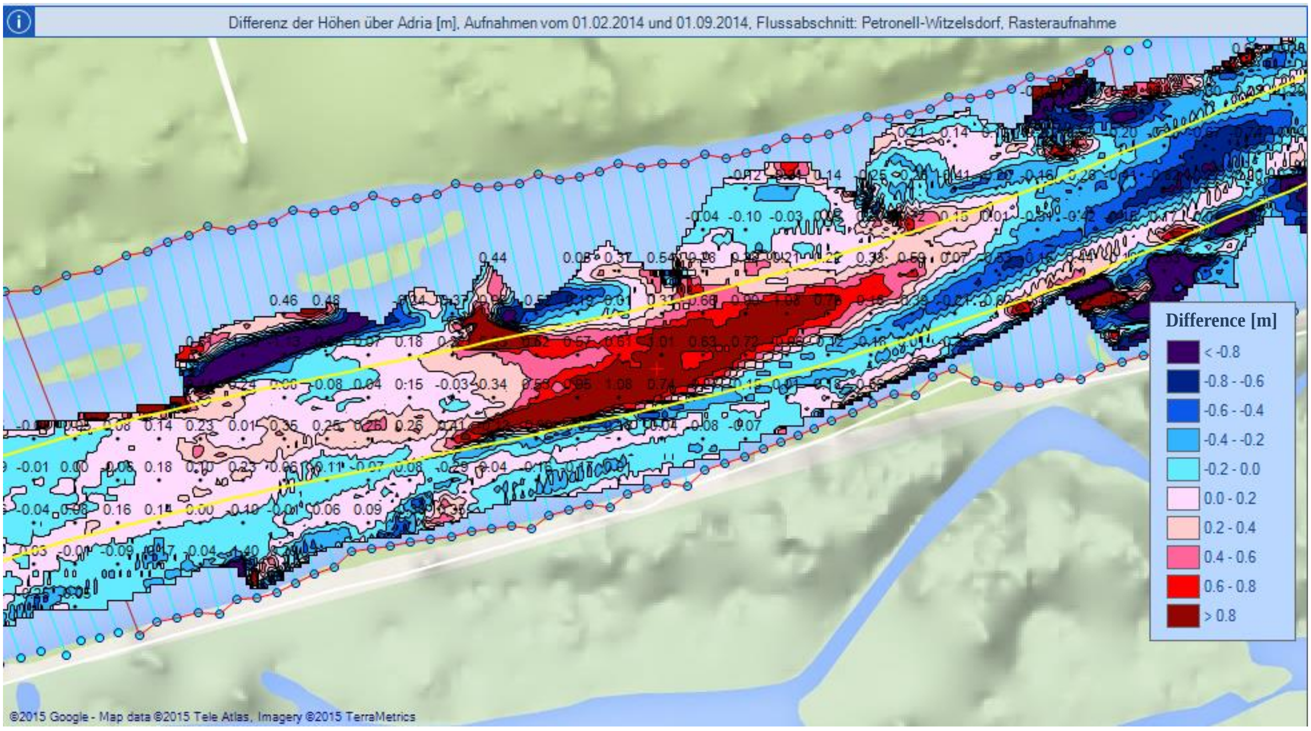


Figure 74: WAMS example - difference map ford Petronell-Witzelsdorf; calculation of contour lines based on subtraction of raster data of regular riverbed surveys carried out on 01.02.2014 and 01.09.2014.

The following figures, Figure 75 to Figure 80, show the genesis of the shallow section Petronell-Witzelsdorf, since autumn 2011, based on TIN-data gained from multi-beam riverbed surveys. Figure 82 illustrates the hydrograph at gauge Wildungsmauer for the period 2011-2015 (development of discharge), which is situated close to the shallow section Petronell-Witzelsdorf allowing for a comparative analysis of sedimentation and discharge level. The presented maps of fairway depths do not show current fairway depths, but depths related to the statistical reference water level LNWL. This means, in turn, that also contour levels are related to this statistical reference water level. This allows for a better comparability of maps showing the condition (altitudes) of the riverbed since water levels change on a daily basis.

The description of the development of the shallow section starts with Figure 75, showing a LNWL-related map of fairway depths based on a TIN-model of Single-beam survey data (06.10.2011) with sufficient fairway depths for the entire fairway. This survey was implemented directly after a dredging measure finished on 4.10.2011.

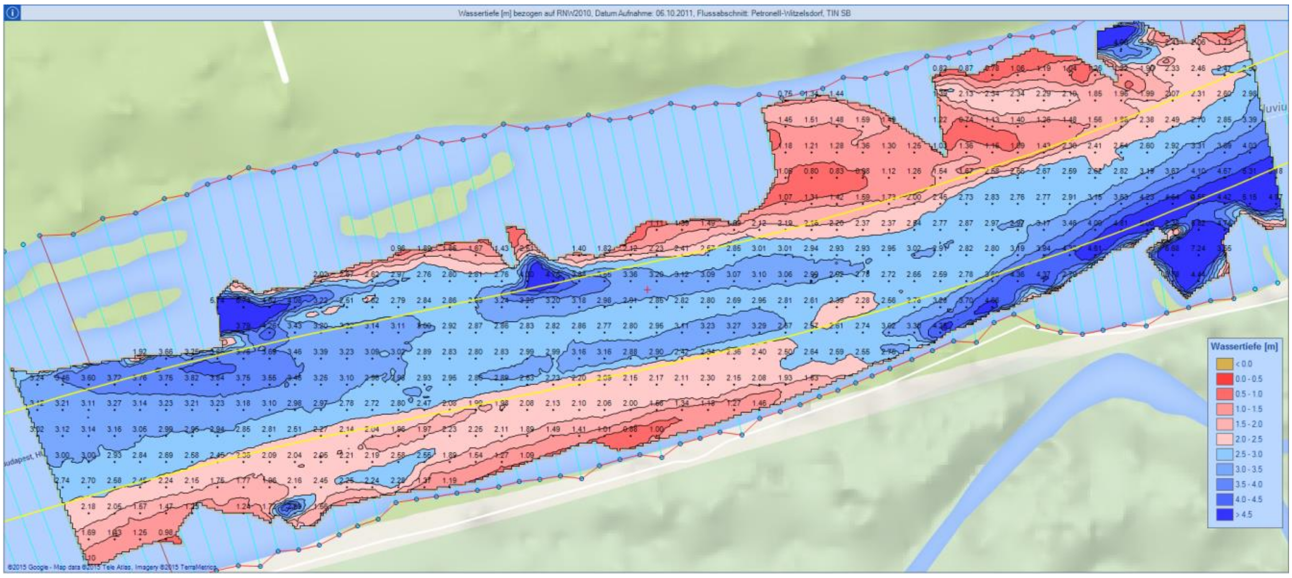
The next map shows a survey implemented on 07.07.2013. During these two years, no dredging activities were conducted. The map clearly depicts that water depths partially fall below 1.2 meter related to LNWL. The difference map was calculated based on equation (48) and shows both the shape and amount of sedimentation occurred during a period of 650 days (around two years), when the survey is compared with the measurement of the riverbed carried out on 07.07.2013. The difference map indicates strong sedimentation in the entire area of the fairway, with elevations exceeding 2 meters at the left edge of the fairway path. This strong sedimentation was influenced by an extreme flood event in June 2013, where the discharge at gauge Wildungsmauer exceeding 10,000m³/s (compare Figure 82). This situation required the implementation of a comprehensive dredging program. Figure 76 confirms that lateral sedimentations were already present in February (05.02.2013), which in turn led to an accretion of the ford which was driven by extreme water levels.

Figure 77 presents the survey of the riverbed (13.8.2013) after the dredging measure which was implemented on 12.8.2013. It is already clearly visible that some shallow areas in the upper area of the shallow section were improved, but downstream additional sedimentations occurred simultaneously. Evidence for the need of further dredging activities is provided by difference map related to the dredging measure.

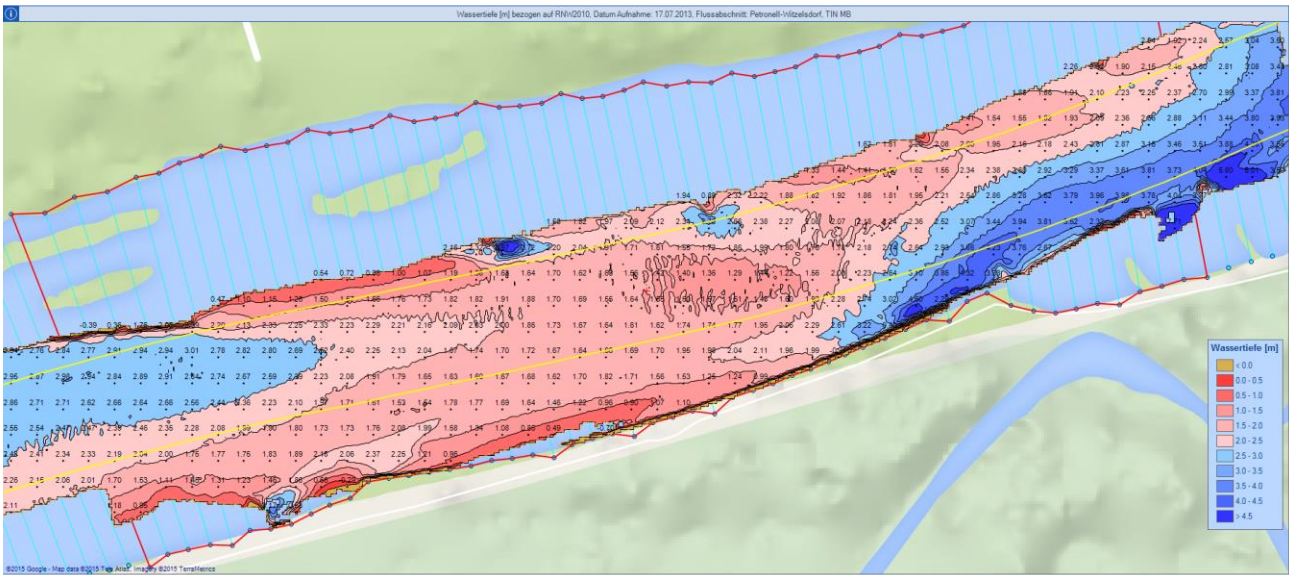
A few days before the MB-riverbed survey was implemented on 13.8.2013, a further dredging measure was started. This dredging measure is illustrated in Figure 78 and was finalized on the 5th of November, 2013. The riverbed survey carried out on the 11th of November already shows significantly improved conditions for inland navigation. For measures with long implementation duration, it naturally comes to the sliding down of sediment and new sedimentations. The difference map clearly depicts the area of dredging. The maximum difference value after the dredging measure amounts to 2.27 meters.

Figure 79 describes the further development of riverbed after the implementation of the dredging measure in November 2013, up to November 2014. Within this year, the strong sedimentation tendency continued, leading subsequently to the situation that the riverbed survey implemented on 12th November 2014 again displayed the fairway depths below 2.5 meters related to LNWL. The difference map clearly indicates that the area, where the dredging measure was implemented was refilled again and additional sedimentation processes occurred downstream. Based on this riverbed survey a new dredging measure started again in December 2014.

Ford Petronell-Witzelsdorf – SB-TIN-data (06.10.2011) – after dredging (04.10.2011); depths related to LNWL 2010



Ford Petronell-Witzelsdorf – MB-TIN-data (17.07.2013) - depths related to LNWL 2010



Difference map based on TIN-data measured on 06.10.2011 and 17.07.2013; visualization of sedimentation and erosion

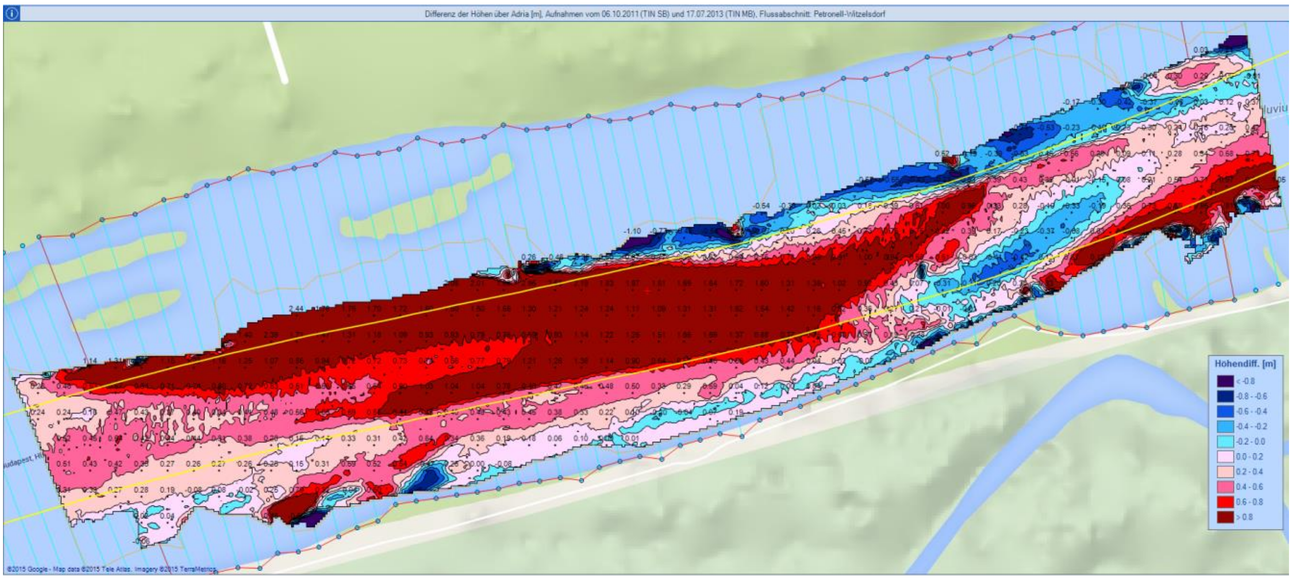
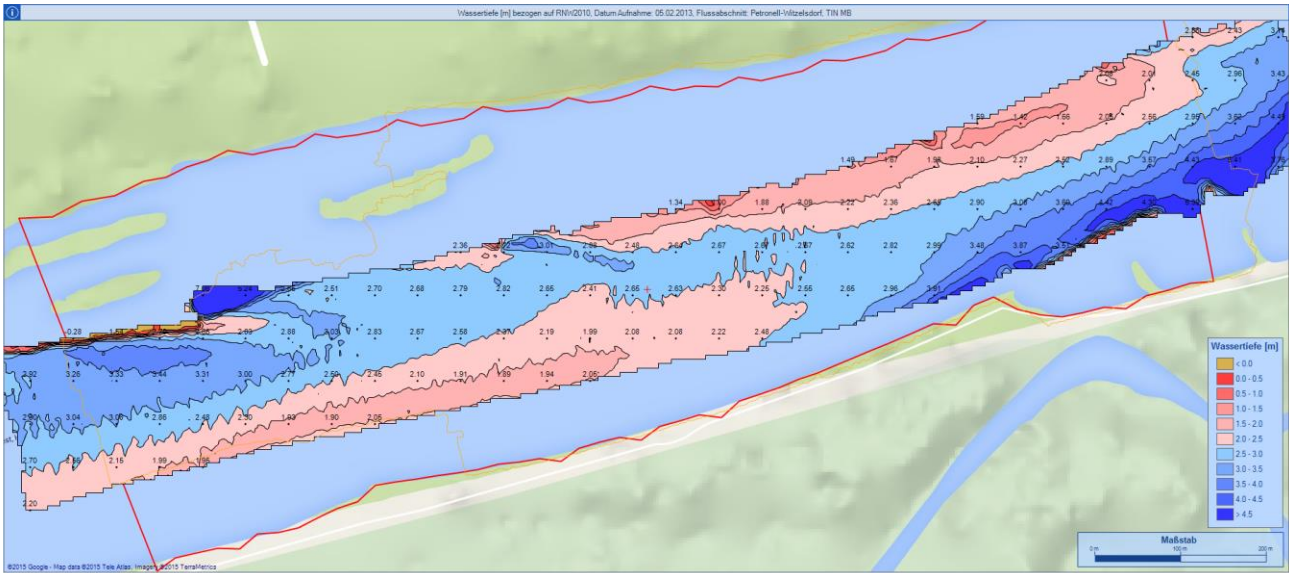
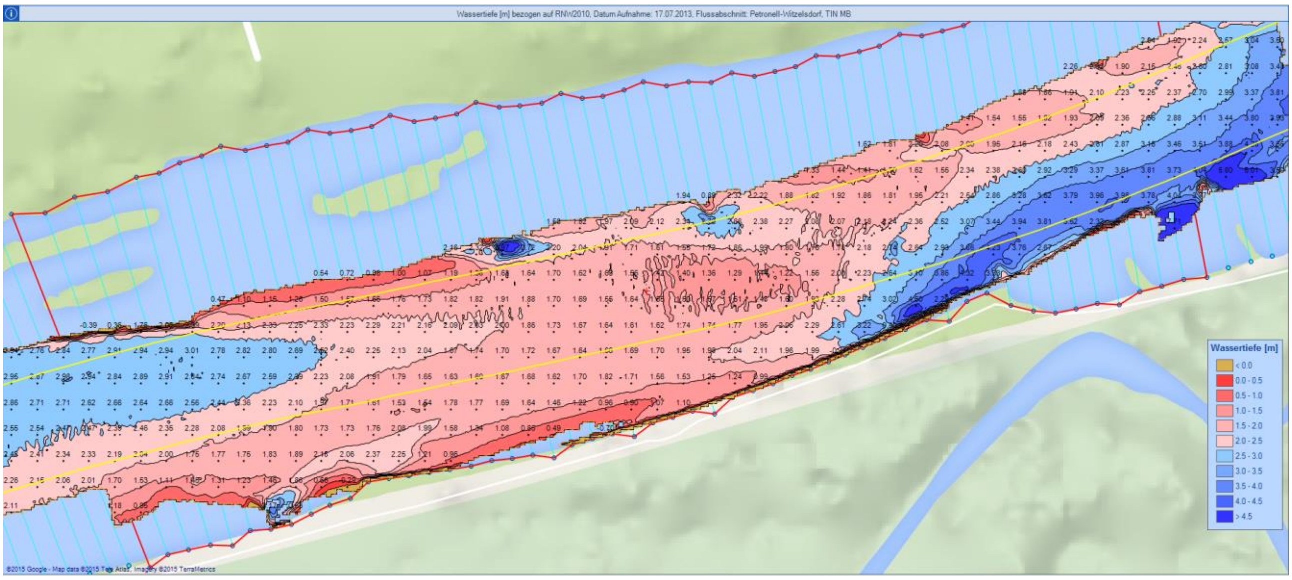


Figure 75: WAMS example – difference map ford Petronell-Witzelsdorf, the model of altitude differences is based on multi-beam TIN-data measured on 06.10.2011 and 17.07.2013 with LNWL as reference level for contour lines.

Ford Petronell-Witzelsdorf – MB-TIN-data (05.02.2013) – before flood event; depths related to LNWL 2010



Ford Petronell-Witzelsdorf – MB-TIN-data (17.07.2013) – after extreme flood event in June; depths related to LNWL 2010



Difference map showing sedimentation due to the flood event in June 2013 based – MB-TIN-data

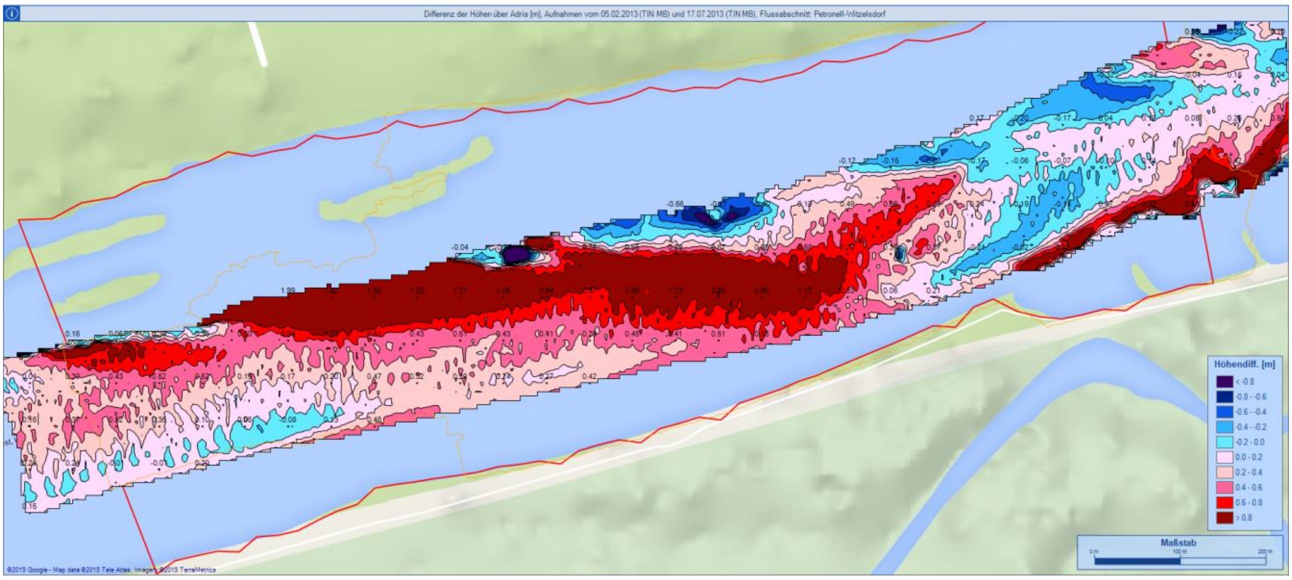
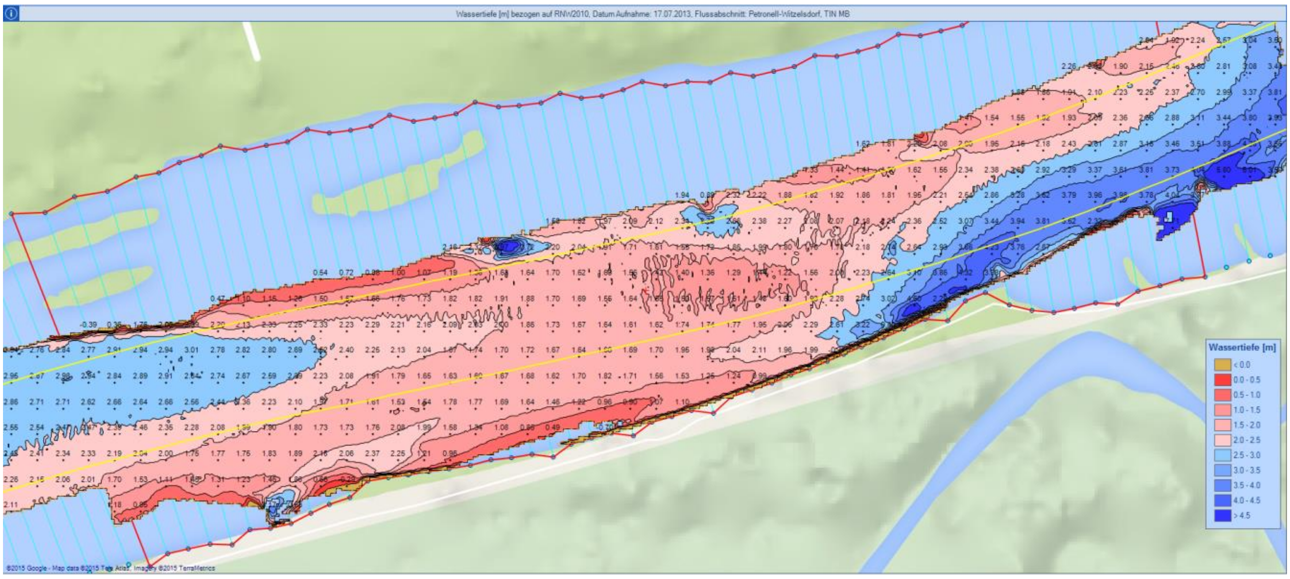
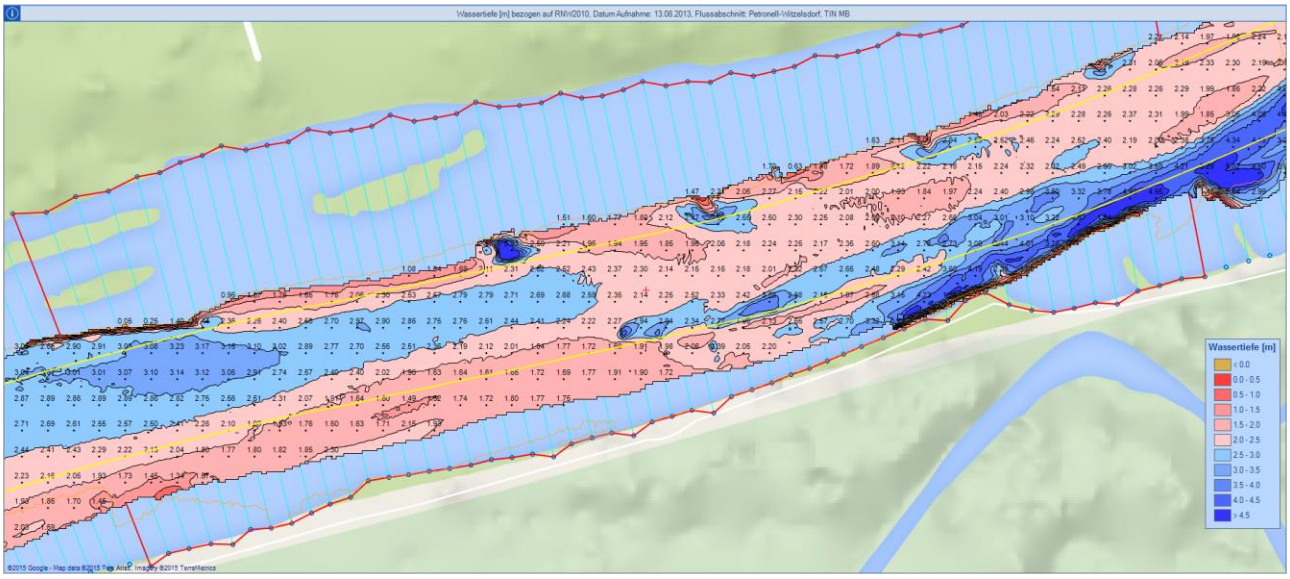


Figure 76: WAMS example – difference map ford Petronell-Witzelsdorf illustrating the sedimentation after the flood event in June 2013, the model of altitude differences is based on multi-beam TIN-data measured on 05.02.2013 and 17.07.2013 with LNWL as reference level for contour lines.

Ford Petronell-Witzelsdorf – MB-TIN-data (17.07.2013) – before dredging; depths related to LNWL 2010



Ford Petronell-Witzelsdorf – MB-TIN-data (13.08.2013) – after dredging; depths related to LNWL 2010



Difference map based on TIN-data measured on 17.07.2013 & 13.08.2013; including dredging finished on 12.08.2013

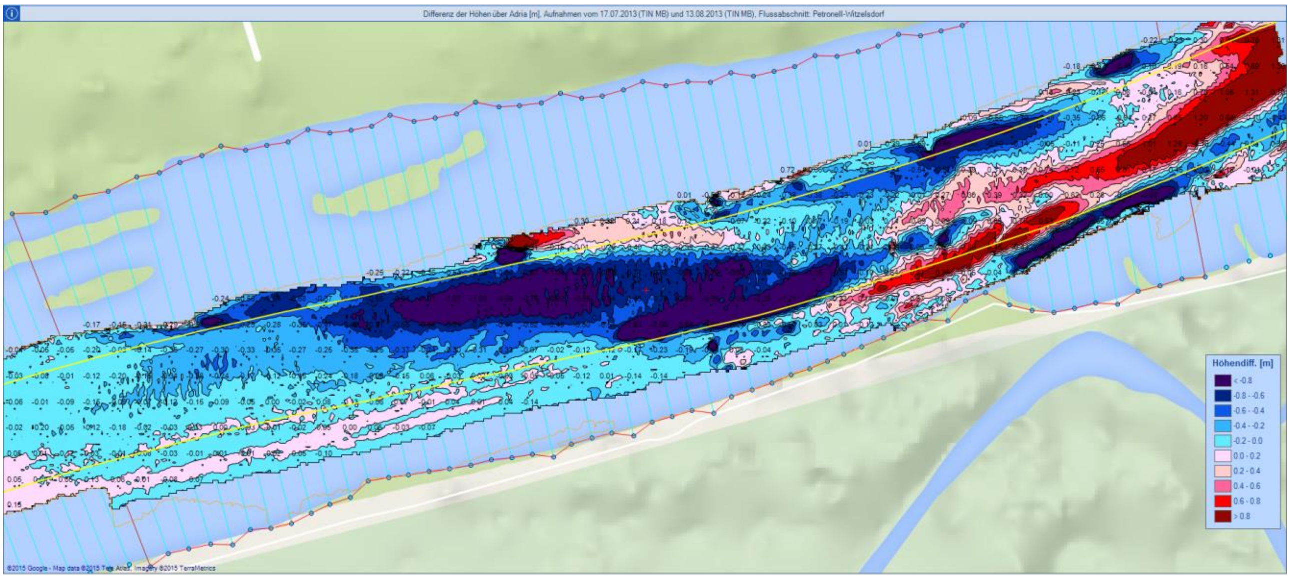
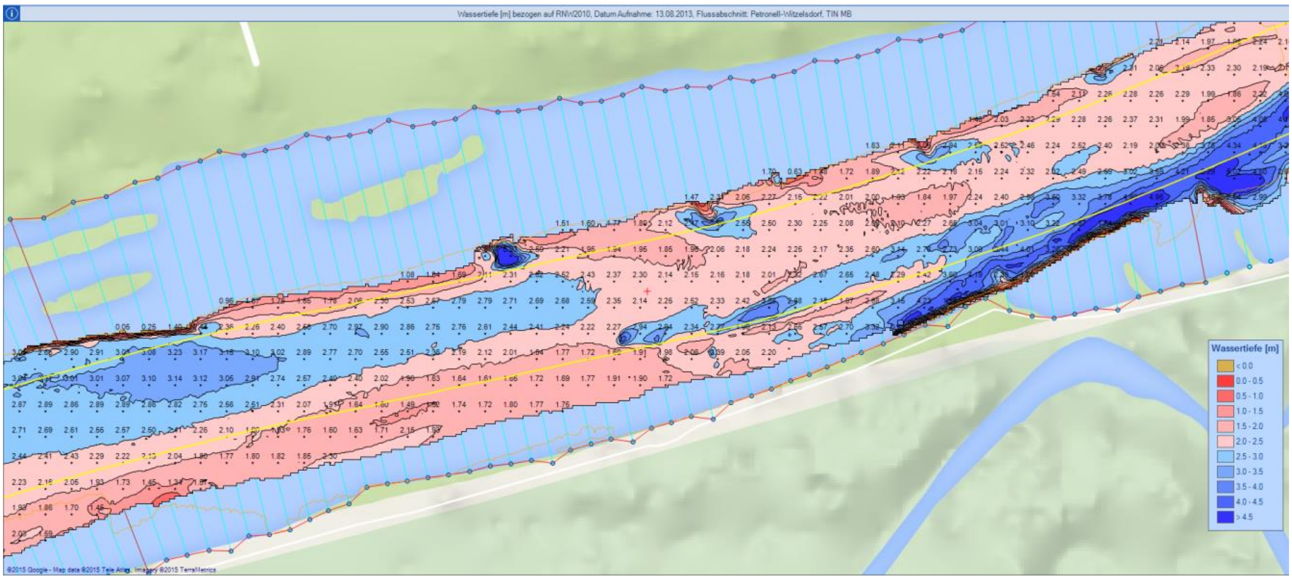
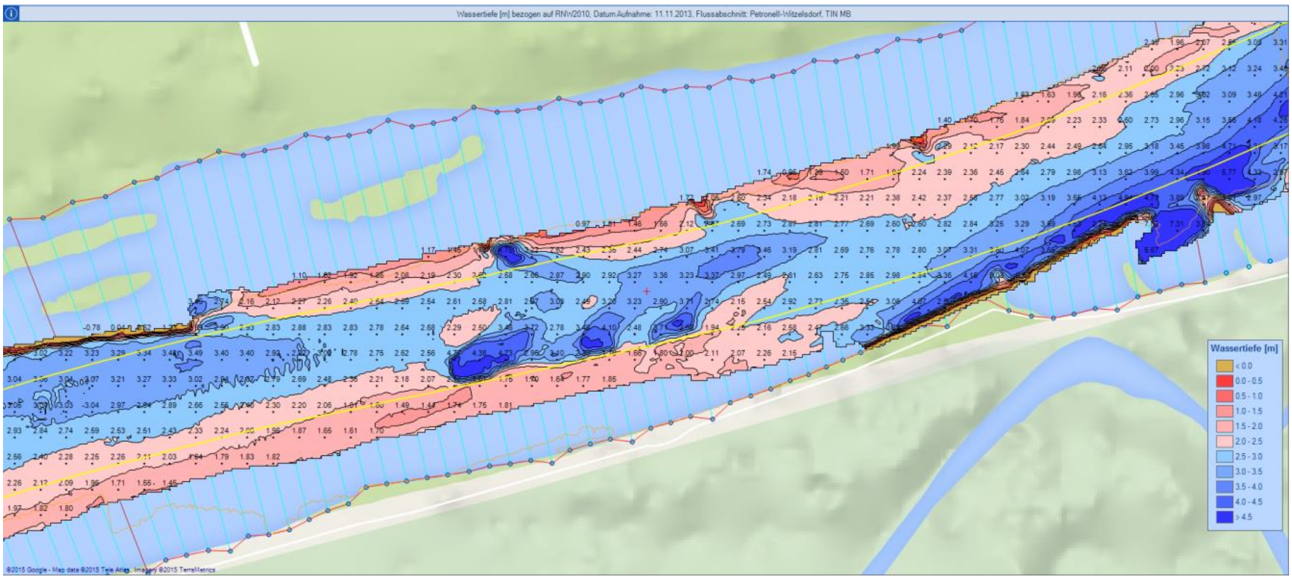


Figure 77: WAMS example – difference map of dredging measure (measure finished at 12.08.2013) at ford Petronell-Witzelsdorf, the model of water depths is based on multi-beam TIN-data measured on 17.7.2013 and 13.08.2013 with LNWL as reference level for contour lines.

Ford Petronell-Witzelsdorf – MB-TIN-data (13.08.2013) – before dredging; depths related to LNWL 2010



Ford Petronell-Witzelsdorf – MB-TIN-data (11.11.2013) – after dredging (05.11.2013); depths related to LNWL 2010



Difference map of TIN-data between 13.08.2013 and 11.11.2013; including a dredging measure finished on 05.11.2013

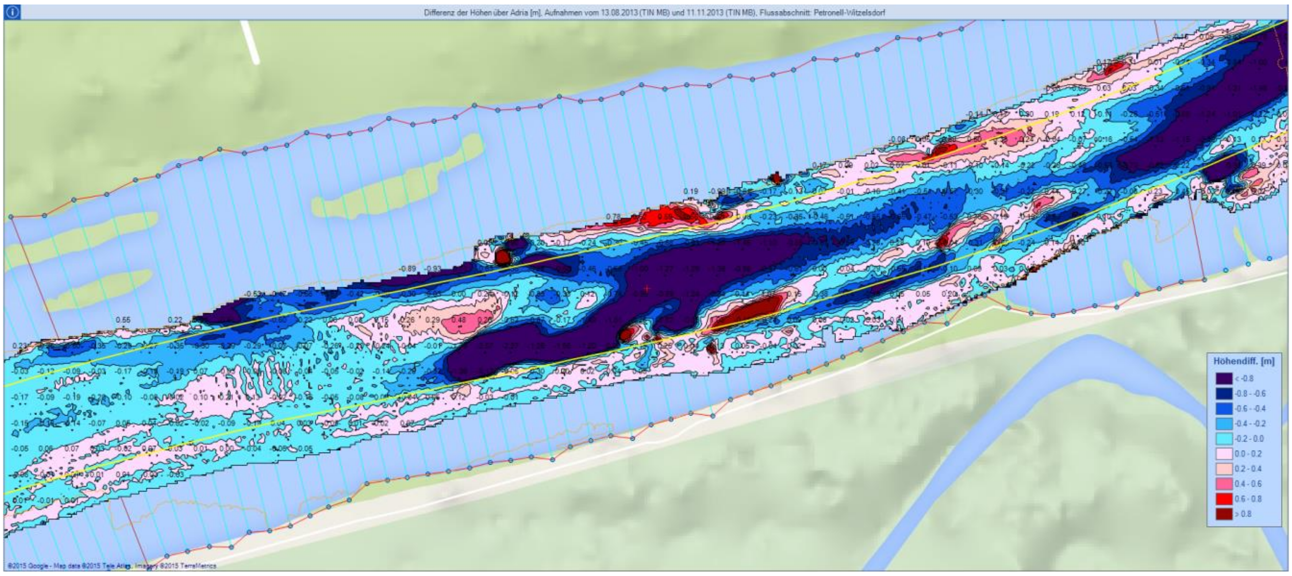
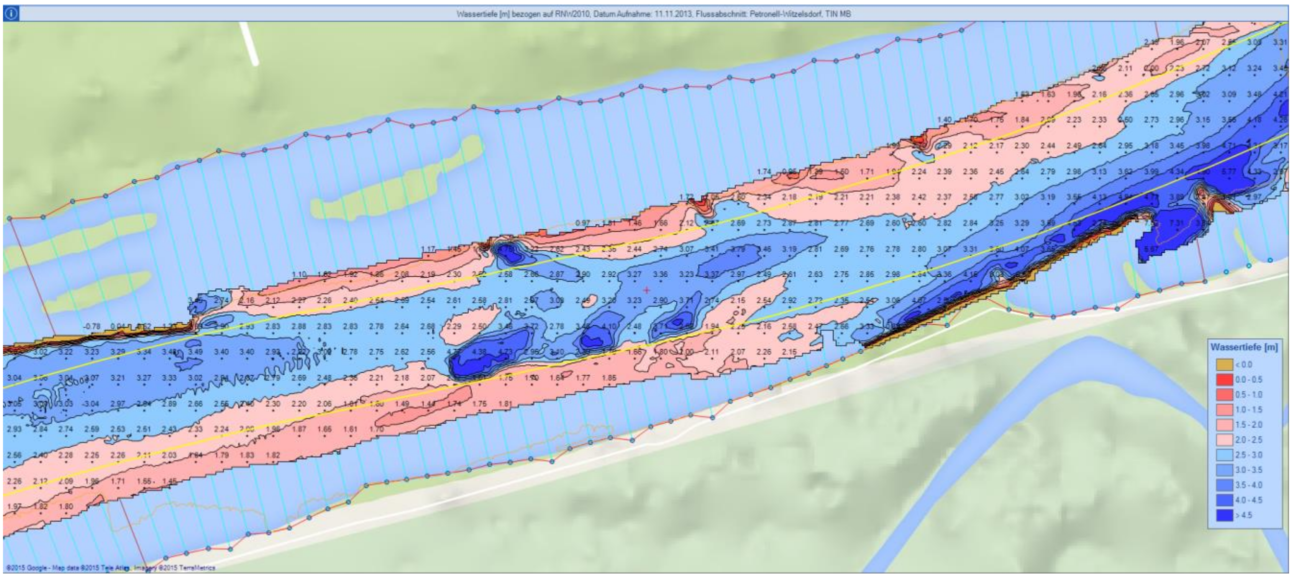
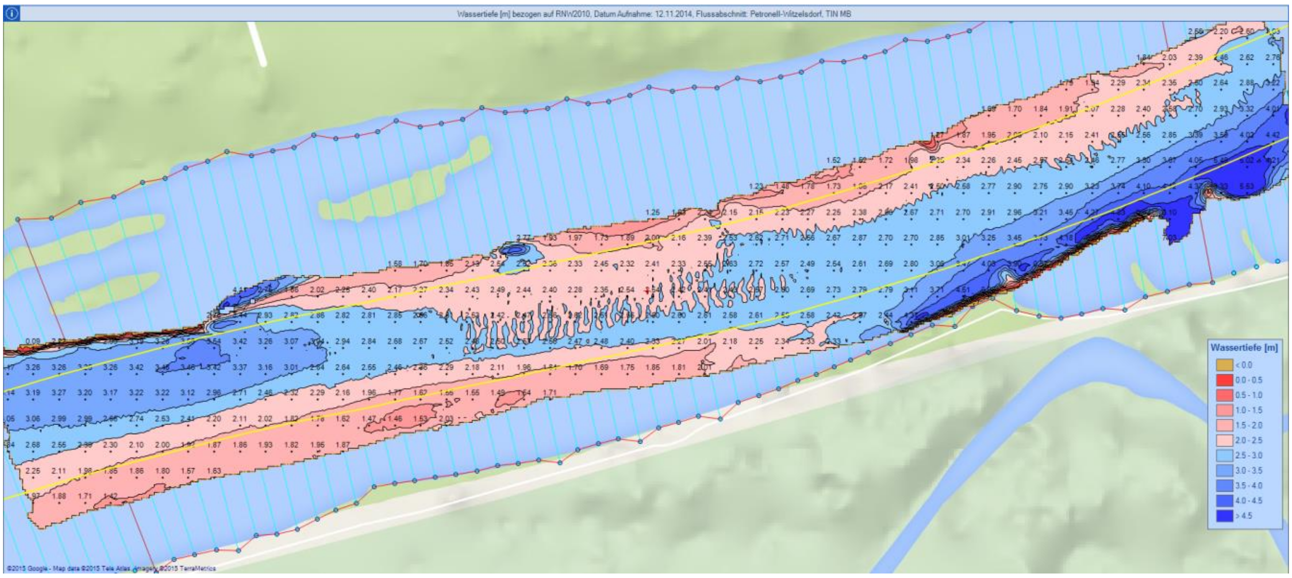


Figure 78: WAMS example – difference map of a dredging measure (measure finished at 05.11.2013) at ford Petronell-Witzelsdorf, the model of altitude differences is based on multi-beam TIN-data measured on 13.08.2013 and 11.11.2013 with LNWL as reference level for contour lines.

Ford Petronell-Witzelsdorf – MB-TIN-data (11.11.2013) – after dredging (05.11.2013); depths related to LNWL 2010



Ford Petronell-Witzelsdorf – MB-TIN-data (12.11.2014) – depths related to LNWL 2010



Difference map based on TIN-data measured on 11.11.2013 and 12.11.2014; visualization of sedimentation and erosion

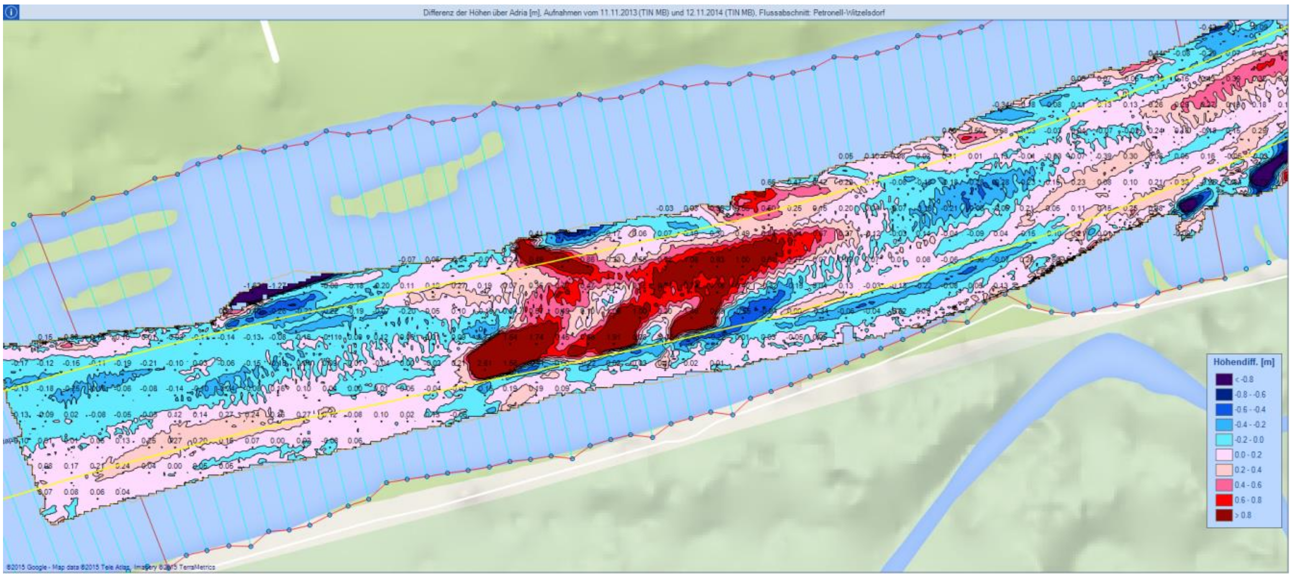
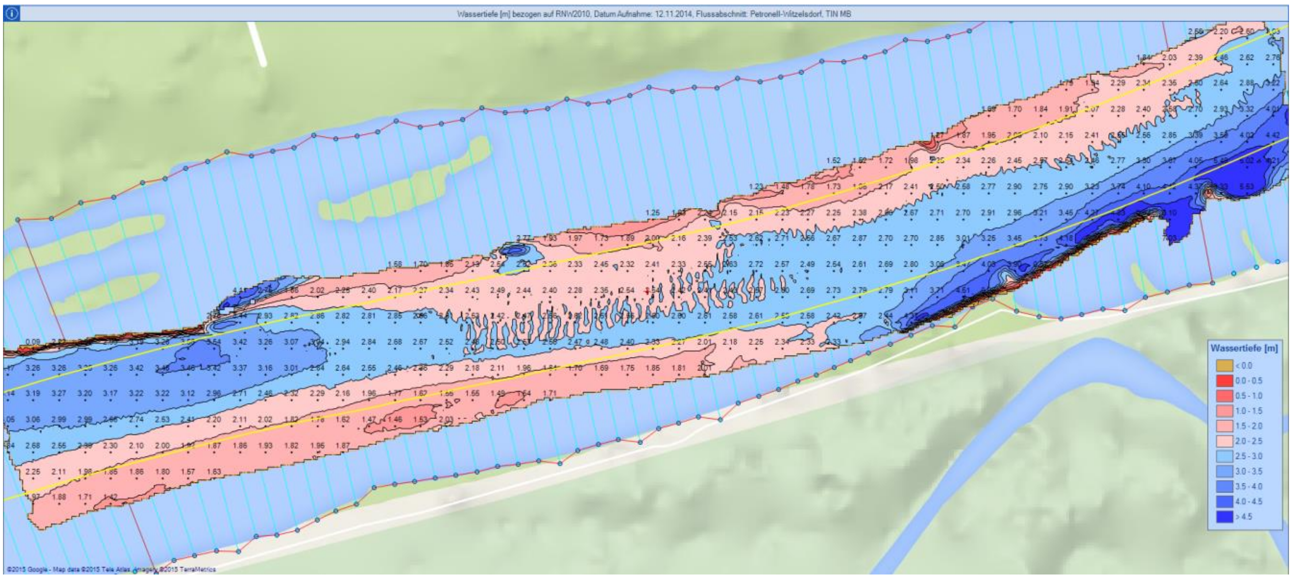
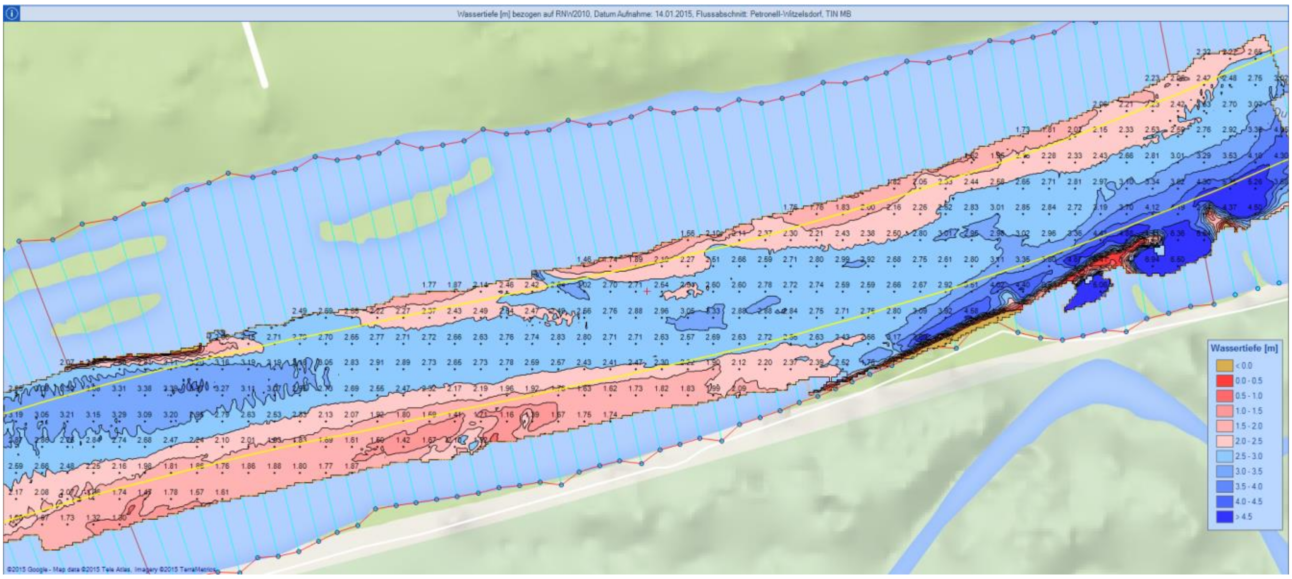


Figure 79: WAMS example – difference map ford Petronell-Witzelsdorf, the model of altitude differences is based on multi-beam TIN-data measured on 11.11.2013 and 12.11.2014 with LNWL as reference level for contour lines.

Ford Petronell-Witzelsdorf – MB-TIN-data (12.11.2014) – dredging started on 04.12.2014; depths related to LNWL 2010



Ford Petronell-Witzelsdorf – MB-TIN-data (14.01.2015) – during dredging; measures finished at 31.01.2015; LNWL 2010



Difference map based on TIN-data between 12.11.2011 & 14.01.2015; including a dredging measure 04.12.2014-31.01.2015

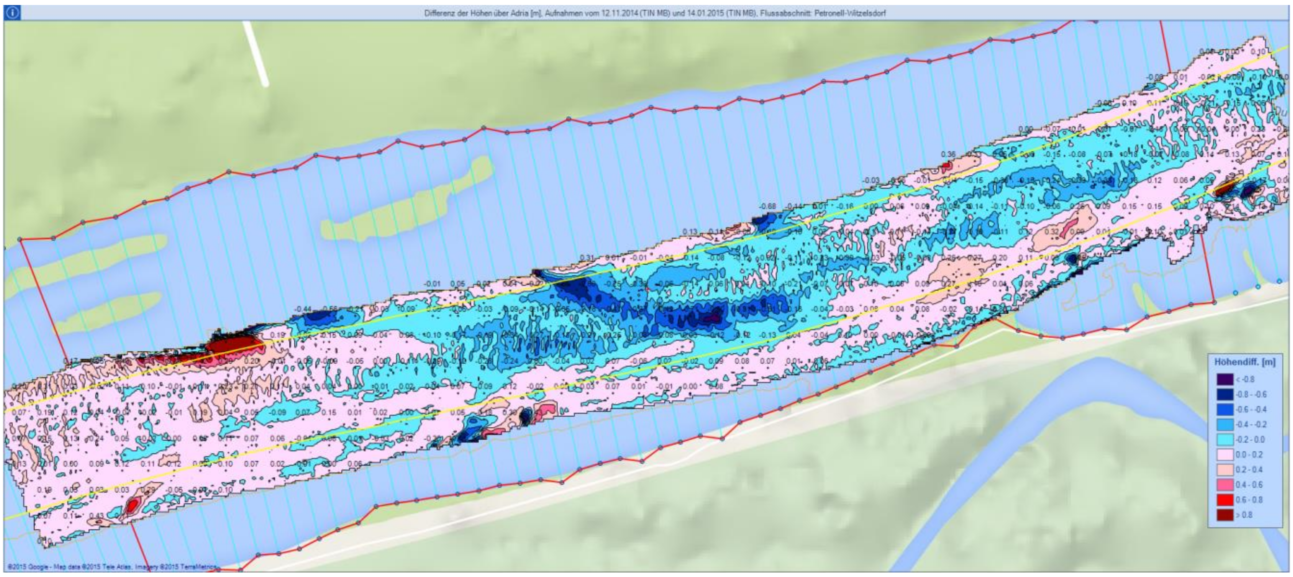


Figure 80: WAMS example – intermediate result of a dredging difference map (04.12.2014-31.01.2015) at ford Petronell-Witzelsdorf; the model of altitude differences is based on multi-beam TIN-data measured on 12.11.2014 and 14.01.2015 with LNWL as reference level for contour lines.

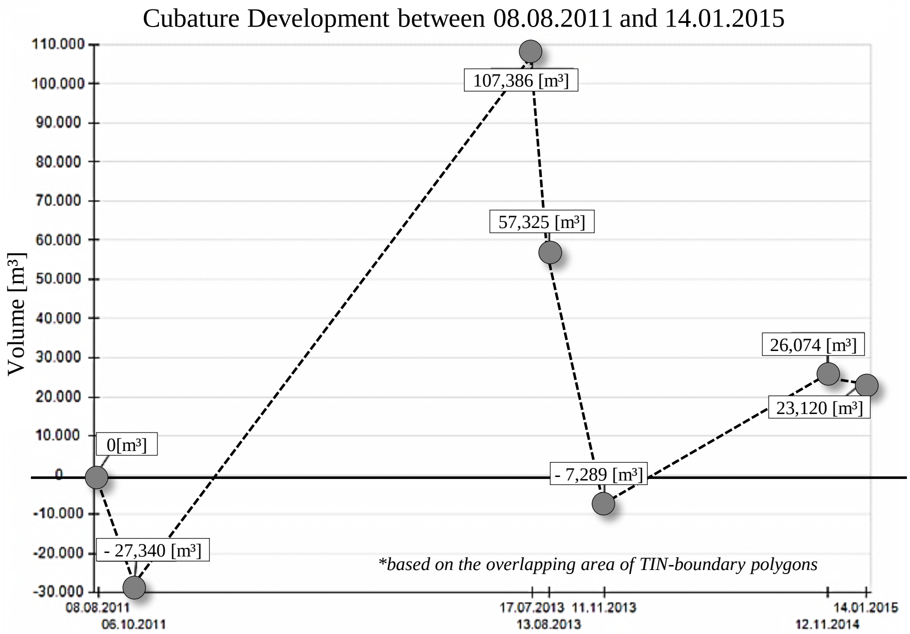


Figure 81: Calculation of cubature development between 08.08.2011 and 14.01.2015

These difference maps allow for the quantification of sedimentation only to a limited extend. A determination of sedimentation or dredging volumes is also possible based on analytical volume calculations, which will be presented in chapter 5.3. Figure 81 illustrates the calculated volume differences between two riverbed surveys for the area, which is mapped by both of the bounding polygons of TIN-data. This method allows for a determination of the extent of sedimentation, erosion and dredging volumes. The figure clearly identifies periods with dredging activities and also periods with strong sedimentation. The calculation of volumes is only possible for those areas of the raster grid with available TIN data for all relevant surveys. Therefore, the smallest bounding polygon determines the area that can be used for any volume calculation. Such calculations of volume differences can be carried out between two surveys with a vector data model using the smaller bounding polygon as a boundary. Other boundary lines for volume calculation may be given by the edges of the fairway, or may be determined by the definition of a dredging polygon.

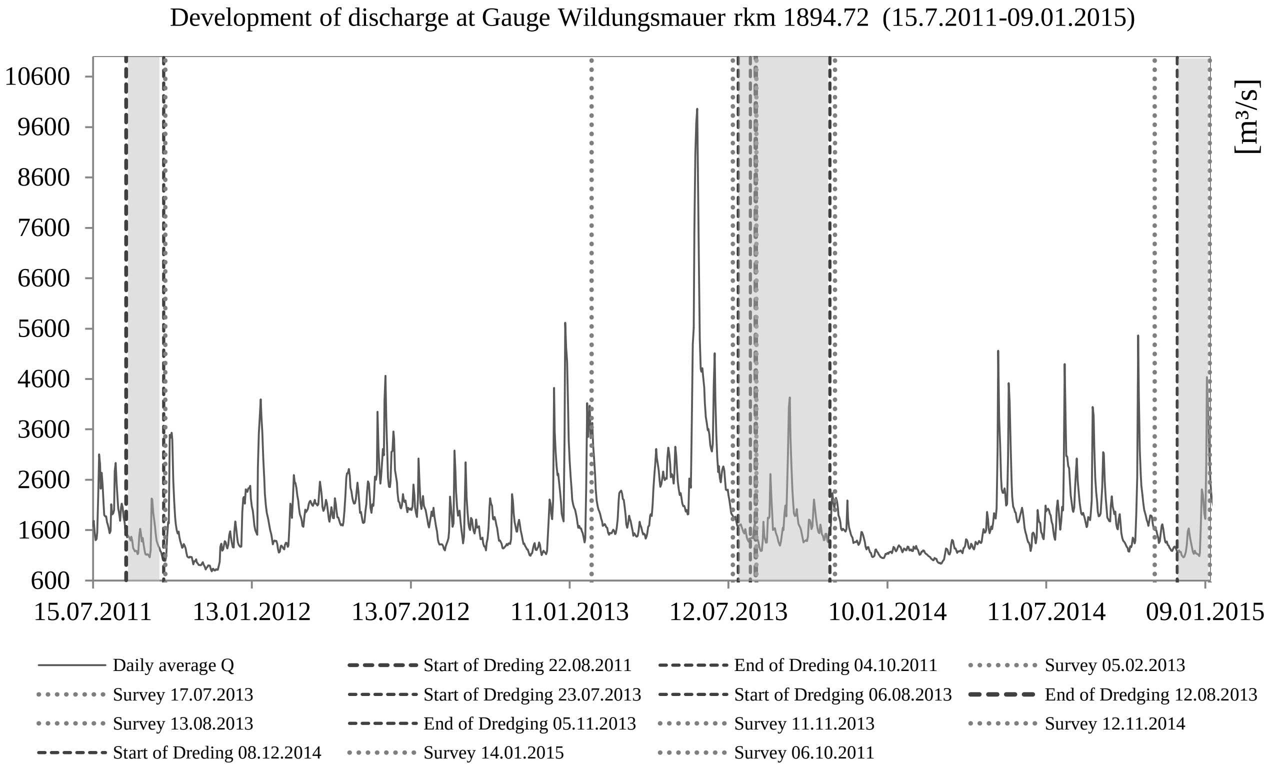


Figure 82: Visualization of discharge data, provided by gauge Wildungsmauer rkm 1894,72 situated closely to ford Petronell –Witzelsdorf (rkm 1898-1898,8) for the period of riverbed analysis showing significant flood and low water periods

Of course, developments of the riverbed may also be visualized in absolute altitudes above the Adriatic Sea Level as 3D-surfaces. Such an illustration can be found in Figure 83. The figure shows the development of altitudes, for example ford Weißenkirchen. The development of the riverbed between the surveys that were implemented on 29^{th} of January 2013, and 22^{nd} of August is based on liner interpolation, as described in previous sections of this thesis.

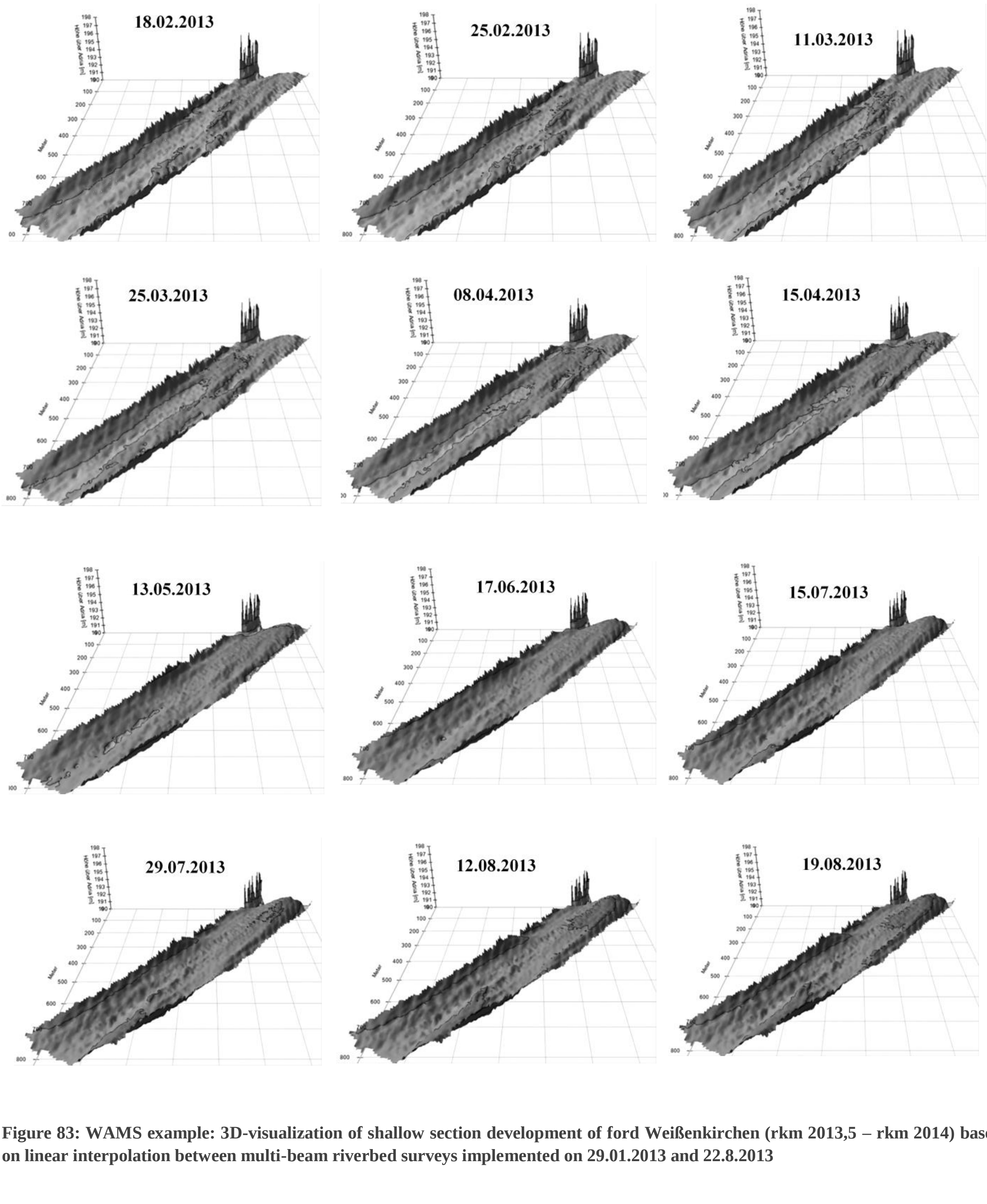


Figure 83: WAMS example: 3D-visualization of shallow section development of ford Weißenkirchen (rkm 2013,5 – rkm 2014) based on linear interpolation between multi-beam riverbed surveys implemented on 29.01.2013 and 22.8.2013

4.3.6 System boundaries and restrictions

Periodic bathymetric surveys of the riverbed and their further processing and modeling, are an import system element of any WAMS. Periodic bathymetric surveys and continuous information from water level gauges provide the necessary data to assess the course of the fairway, draw 2D plans of water depths as well as a 3D model of the fairway together with a calculation of fairway availability. Depending on whether only maps of fairway depths have to be created or volumes of sedimentation processes have to be calculated, different requirements regarding data type (raster, vector) and data density (multi-beam or single-beam) and accuracy must be met. Depending on available equipment, available resources and the accuracy of necessary information there are mainly three typical types of bathymetric surveys.

The first type is longitudinal profile measurements mainly parallel to the fairway path with single-beam equipment that is mainly used for a fast preliminary check of possible changes in the riverbed. A systematic assessment of river morphology, fairway availability and measure impacts, as required in a waterway asset management, is not recommended based only on this approach.

The second types are systematic single-beam measurements of cross-sectional profiles with a fixed distance between 25 and 100 m. Obviously, this type of measurement provides no depth information between the profiles, which may be an advantage regarding the amount and processing of data. Regarding a full WAMS, it is possible to calculate the fairway availability and establish a rough model for an assessment of measure impact based on a very limited amount of data. However, for an in-depth analysis, the resulting accuracy of single-beam measurements is rather low, thus posing a certain limit for any WAMS.

The third type of measurements is multi-beam bathymetric surveys based on overlapping stripes and it provides very accurate dense information of the entire riverbed surface. The high density of measurement data, resulting in a few gigabytes of data per measurement of short shallow sections, is not easy to handle without special software and training. Pre- and post-processing of all data, both from single-beam and multi-beam surveys, is a time-consuming task that is mainly performed manually case-by-case in all waterway agencies, even though the principal input of data and output of plans, maps and analysis stay the same.

Except for the Austrian WAMS showing very promising results regarding fast pre- & post-processing of the resulting big data volumes there are currently no real WAMS solutions on the market. As a result there are almost no common standards regarding the storage, processing and analysis of bathymetric data being mainly organized depending on different software products and perceived national practical needs. With the common goal of providing accurate information on fairway conditions as a basis for the planning of measures and navigational purposes, the question of the necessary means to obtain this information arises. As a first approach for an assessment of these means, a periodic survey of the entire riverbed at least once a year and an additional 5 surveys of all critical sections are defined. Based on actual knowledge and practical experience, such a density of surveys would fulfill all possible needs for analysis as well as a full WAMS.

Many waterway operators use special software solutions or in-house developed software solutions in their technical departments as hydrology and hydrography, with separate databases for data processing and analysis. However, waterway asset management requires a cross-analysis of all of these data, which is essentially based on the merging of databases into a holistic WAMS database. WAMS as software is not considered as a competitor to conventional geographic information systems, but can interact with them via interfaces, cross-process and evaluated data to provide, in turn, the basic information for management decisions.

4.4 Fairway modelling

In waterway asset management, the fairway is the third major system element, besides the water level model and the riverbed model, for building a condition model of the fairway path as the center part of inland waterways as a transport mode. The fairway is a defined area of the cross section of a river showing a certain width and depth that may be marked with buoys at shallow or narrow sections of the river. The fairway axis must not compulsorily be located along the river axis. Usually, the fairway channel (fairway with an associated depth) is located in cross-sectional areas of a stream with the greatest water depths. Thus, evidence is given that with increasing fairway width, it is more challenging to find an area within the cross section that offers sufficient depths on the entire fairway width.

Fairway width is an important parameter that defines which size of vessels or vessel-formations are able to operate on the respective river section, and whether encounters or overtaking maneuvers are possible. In addition to the width of fairway lanes, certain safety clearances are necessary, both between the two fairway lanes and between the fairway lanes and the river banks. These safety clearances further depend on convoy size, vessel speed, flow velocity and curve radius. Overtaking maneuvers generally require higher distances between vessels than standard vessel encounters (compare equations (49) and (50)) [Tschernutter P. 2012]. The resulting water width at vessel floor level w_w between fairway lanes for the encounter of two individual vessels, each with a certain width w_i, is calculated based on the equation (see Figure 84):

$$w_w \geq 1,4 \cdot \sum w_i \tag{49}$$

For overtaking maneuvers, the necessary safety clearance between fairway lanes is calculated based on the following equation (compare Figure 85):

$$w_w \geq 2,2 \cdot \sum w_i \tag{50}$$

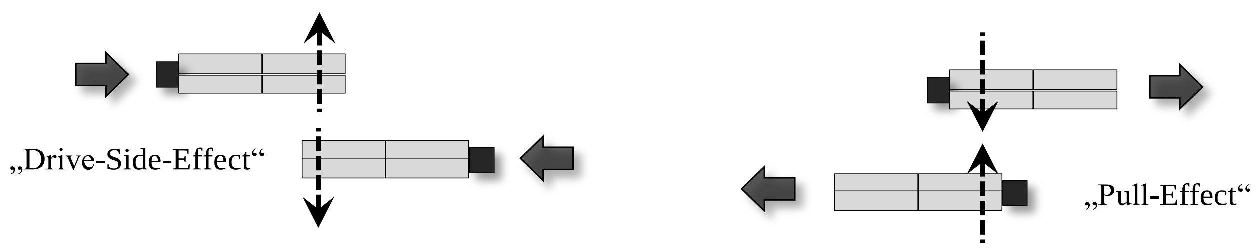


Figure 84: Force-effects occurring during an encounter scenario of two convoys providing the basis for the derivation of safety clearances [Tschernutter P. 2012]

Figure 85: Force-effects occurring during an overtaking maneuver involving two pushed convoys providing the basis for the derivation of safety clearances [Tschernutter, P. 2012]

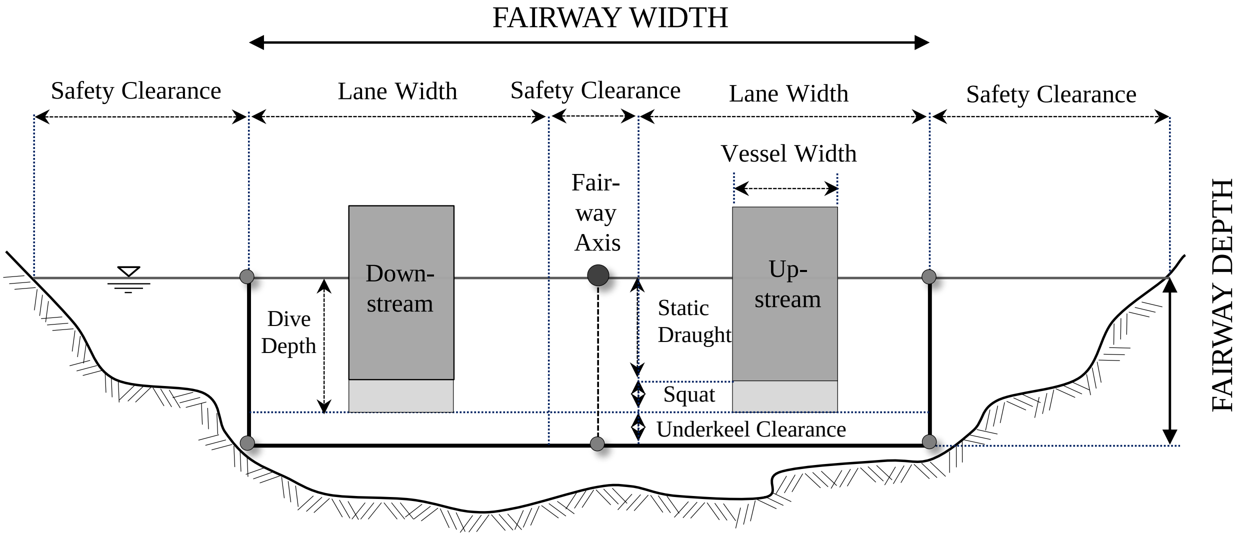


Figure 86: Definition of fairway parameters for a cross-sectional profile. The illustration determines both fairway width and fairway depth, which consists of static draught, dynamic squat, and underkeel clearance; own illustration based on [VIADONAU 2013b]

Fairway depth, as defined in Figure 86, describes an even more important parameter for inland waterways, which consequently defines how much freight individual vessel types or convoys are able to load. In order to allow cross-border transport covering entire rivers, it was necessary to create uniform standards for international waterways, which will be described in below. Figure 86 and Figure 87 provide definitions of fairway width and fairway based on a cross sectional profile and the top view of a river section. The following section in this chapter will provide information about currently valid agreements on fairway parameters for the Danube, reflections regarding possible definitions of fairway categories (levels of service) and the fairway approach used in the WAMS-software.

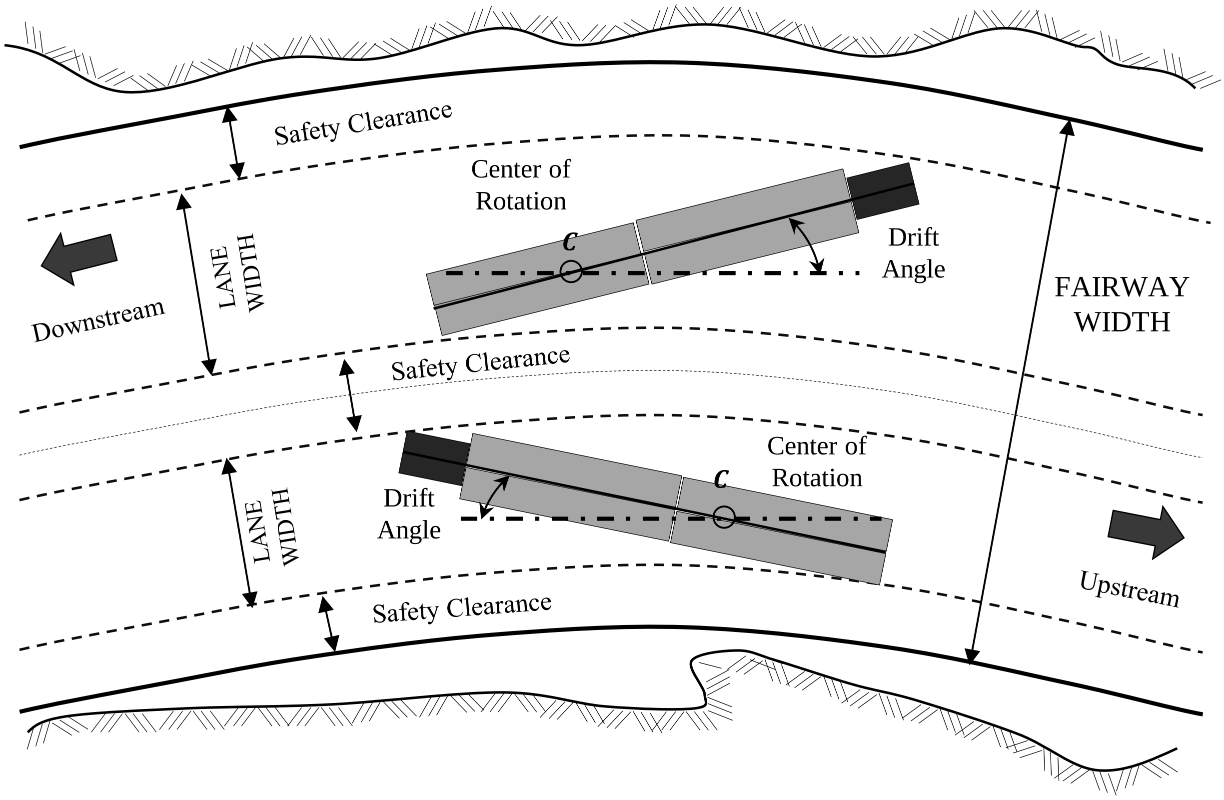


Figure 87: Top view of a curved river section, determining the fairway width required for the encounter of two pushed convoys with four barges including necessary safety clearance

4.4.1 Existing recommendations and agreements

In order to make inland waterway transport in Europe more efficient and attractive to customers, a common legal framework was established in 1996 for the planning of the development and maintenance of the European inland waterway network as well as for ports of international importance based on technical and operational parameters. This "European Agreement on Main Inland Waterways of International Importance" (AGN) classifies inland waterways on the basis of minimum requirements regarding standardized horizontal dimensions of motor vessels, barges and pushed convoys with categories ranging from I to VII. By ratifying this agreement, the contracting parties express their intention to implement the coordinated plan for the development and construction of the so-called E waterway network. Within this waterway framework, the Danube is designated as waterway E 80. The waterway classes following the AGN are illustrated in Figure 88. Waterways of class IV or higher are considered to be of economic importance for international freight transport. The class of an inland waterway is thereby determined by the maximum dimension of the vessels which are able to operate on this waterway. For the upper Danube, the minimum vessel draught should not fall below 2.50 m on at least 300 days per year. The minimum height under bridges related to the highest navigable water level must be given with 5.25 m [UNECE, 2012 and VIADONAU, 2013b].

Motor Cargo Vessels						
Type of Vessel: General Characteristics						
Water -way Class	Designation	Max. Length L(m)	Max. Width B(m)	Static Draught d (m)	Deadweight T (t)	Min. Height Under Bridges H(m)
IV	Johan Welker	80-85	9.5	2.5	1,000-1,500	5.25/7.00
Va	Large Rhine Vessel	95-110	11,4	2.5-2.8	1,500-3,000	5.25/7.00/9.10
Vb	Large Rhine Vessel	95-110	11,4	2.5-2.8	1,500-3,000	5.25/7.00/9.10
VIa	Large Rhine Vessel	95-110	11,4	2.5-2.8	1,500-3,000	7.00/9.10
VIb	Large Rhine Vessel	140	15.0	3.9	1,500-3,000	7.00/9.10
VIc	Large Rhine Vessel	140	15.0	3.9	1,500-3,000	9.10
VII	Large Rhine Vessel	140	15.0	3.9	1,500-3,000	9.10

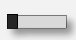
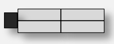
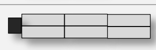
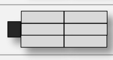
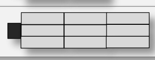
Pushed Convoys						
Type of Convoys: General Characteristics						
Water -way Class	Designation	Max. Length L(m)	Max. Width B(m)	Static Draught d (m)	Deadweight T (t)	Min. Height Under Bridges H(m)
IV		85	9.5	2.5-2.8	1,250-1,450	5.25/7.00
Va		85-110	11.4	2.5-4.5	1,600-3,000	5.25/7.00/9.10
Vb		172-185	11.4	2.5-4.5	3,200-6,000	5.25/7.00/9.10
VIa		95-110	22.8	2.5-4.5	3,200-6,000	7.00/9.10
VIb		185-195	22.8	2.5-4.5	6,400-12,000	7.00/9.10
VIc		270-280	22.8	2.5-4.5	9,600-18,000	9.10
VIc		195-200	33-34.2	2.5-4.5	9,600-18,000	9.10
VII		275-285	33-34.2	2.5-4.5	14,500-27,000	9.10

Figure 88: Waterway classes according to the AGN

Further important recommendations for a uniform navigability of the Danube were already specified in 1948. With the Belgrade Convention, the Danube Commission (DC) was established as an intergovernmental organization with the task of supervising the implementation of the provisions of the Convention with regard to the regime of navigation on the Danube. Specifications of fairway widths and depths were implemented as a function of riverbed material and result, e.g. for the Austrian stretch of the Danube waterway, in a fairway width for free-flowing sections of 120 m (Wachau) and 75 m resp. 120 m (east of Vienna) at a recommended fairway depth of 2.5 m to be reached on at least 343 days per year (94% of the year) [VIADONAU 2013b].

In practice, these recommended fairway parameters, especially a fairway width of 120 m, can hardly be met due to the hydro-morphological situation on these stretches as well as economic and environmental issues (restrictions according to environmental law permits, national park “Donau Auen”). Studies on the cost-effectiveness of these recommendations, regarding their impacts on navigation companies and waterway authorities, are only at the beginning stages. In Austria, a first analysis of fleet composition and traffic volumes on the upper Danube (traffic throughput at a lock of hydropower plant Altenwörth) provided evidence for the claim that critical vessel encounters on narrow river sections are hardly a problem. The probability of a critical encounter of two 4-unit pushed convoys (pusher plus four barges as maximum configuration on the Austrian stretch of the Danube) on the narrow sections is rather low due to the fact that their share of the vessel fleet, with an average total of 19 vessels per day amounts to only 4.2% [Hoffmann M., Haselbauer K. & Blab R. 2014a].

4.4.2 The concept of Service Levels (LOS)

Further analysis of transponder data proves the low utilization of fairway width, which is – with the exception of the German stretch of the waterway – therefore not a critical issue on the river Danube given the current transport volume. The possible draught load of vessels, however, significantly influences the transport costs on the Danube and the competitiveness of this mode of transport.

Since the aspect of economic efficiency for all modes of transport is becoming increasingly important, it is also necessary to assess the utilization of fairway widths for inland waterways and, thus, evaluate whether available fairway widths are required in terms of actual traffic amounts and transport volumes. For waterway administrations, the implementation of the concept of levels of service means, that waterway operation in the case of low utilization and low water conditions, becomes economically more efficient due to reductions of necessary dredging volumes.

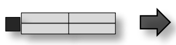
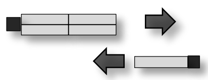
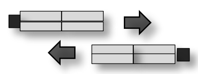
	Level of Service 1 (LOS 1)	Level of Service 2 (LOS 2)	Level of Service 3 (LOS 3)
			
Curve Radius	Pushed Convoy with 4 Barges One-Way Traffic Downstream	Encounter of a Pushed Convoy with 4 Barges (Downstream) & a Single Vessel (Upstream)	Encounter of two Pushed Convoys with 4 Barges
< 1000 m	80	120	160
1000 -1500 m	60	100	120
> 1500 m	40	80	100

Figure 89: Recommendations of necessary minimum fairway widths referring to Levels of Service based on critical encounter cases and curve radius [Research center for inland vessel construction Duisburg]

As a part of internal quality reviews at VIADONAU three fairway categories (Levels of Service) were defined for the Austrian river stretch based on a study of the research center for inland vessel construction in Duisburg, which dealt with the particular space requirements of different vessel configurations for varying curve radii. The space requirements, especially fairway width, of single vessels and convoys increase significantly when driving through curves. These requirements regarding fairway width are considerably higher in curves than in areas with a straight fairway path and strongly increase with decreasing curve radius. In the study from the research center in Duisburg, the required width was evaluated based on practical experiments for different configurations and vessel sizes on characteristic stretches of the Austrian Danube, considering a vessel speed of 14 km/h and a flow velocity of 6 km/h.

The main operating conditions, consequently defined as levels of service, were selected in a way that they cover all practical relevant conditions. The encounter of two pushed convoys with four barges (LOS 3) is assumed as the largest relevant operation condition, determined by the dimensions of lock chambers. Under low water conditions, the impossibility of the encounter of large convoys in some curve areas is accepted and single-lane traffic is executed (LOS 1). Between single-lane traffic and the operation of the entire fairway width, level of service 2 (LOS2) allows the encounter of a pushed convoy with four barges downstream and a single vessel upstream. Since space requirements downstream are much higher (drift), this case was considered as decisive to the definition of fairway widths for single-lane traffic. For a curve radius of < 1,000 m, fairway widths are 80 m (LOS 1), 120 m (LOS 2) and 160 m (LOS 3) respectively, all for a common fairway depth of 2.5 m (compare Figure 89) [Research center for inland vessel construction Duisburg].

4.4.3 Applied fairway model

The following section will highlight the model of the fairway channel as it is an important part of each waterway asset management system. A fairway path may be either be created directly using the WAMS-software or be imported from other software solutions such as GIS. The developed fairway model is currently closely associated with the cross-sectional profiles and hectometers. Hectometers are mounted on both riverbanks of the Danube with a distance of 100 meters visualized by labeled blocks of stone. The coordinates of hectometer points are known as well. The respective hectometer points on the left and the right bank of the river are now connected, forming cross-sectional profiles as shown in Figure 90. Furthermore, the endpoints of cross-sectional profiles, which are located with a distance of 50 meters in the middle between hectometer profiles, are known by coordinates. By linear interpolation between the end points of hectometer cross sections and cross sectional profiles with a distance of 50 meters, the coordinates of the endpoints of cross sectional profiles with a distance of 25 meters can be calculated. To set up a model of the fairway channel, any fairway, with a known left and right border line may be imported. In the next step, the two boundary lines of the fairway (the left and the right border line) are cut with cross-sectional profiles. These points of intersection are stored coordinately in the fairway database. Afterwards, the fairway axis is determined as the center line of the boundary lines. Together with the coordinates of the mid points of the fairway axis, the distances to the left and right border lines of the fairway are stored.

Using these coordinates, the fairway is given in its horizontal position. The altitudes of the left and right fairway borders, and the fairway axis for this cross sectional profile, may either be given by the altitude of the statistical low water level line or the current water level line. Based on the altitude of these water level lines, the fairway depth (bottom of the fairway) can be defined as a difference value, either with 2.5 meters or with any other target depth. Using this method, all levels of service (LOS1, LOS2, LOS3) can be modeled in waterway asset management. Based on the fairway axis and the two border lines of the fairway, which are furthermore associated with fairway depths, any operational modification of the fairway may be implemented. These modifications include, for example, narrowing, widening or shifting of the fairway based on either operational reasons, such as sunken vessels, or maintenance reasons such as insufficient fairway depths in the initial fairway area. The illustration options of the fairway and the possibilities of modifying location and

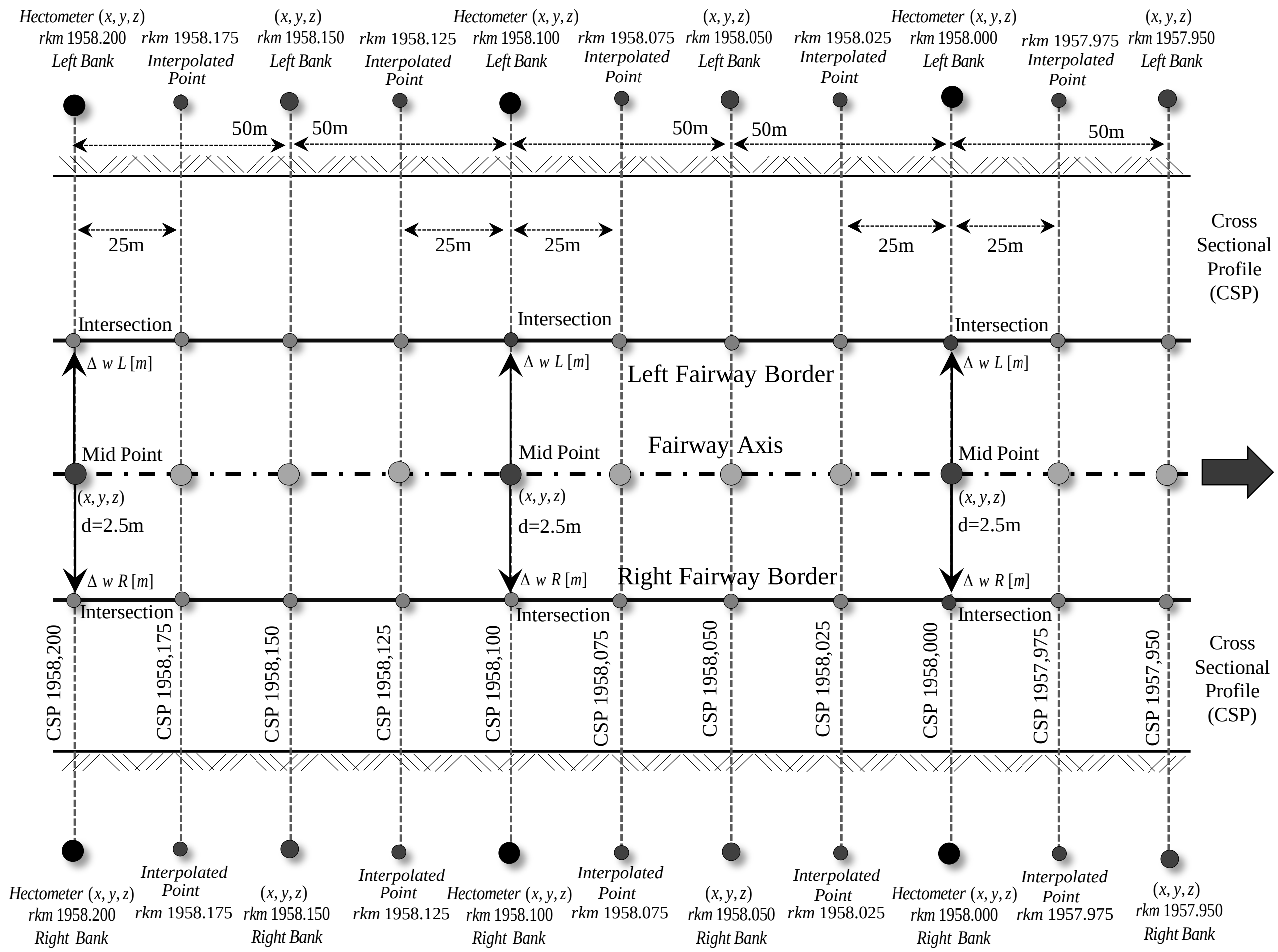


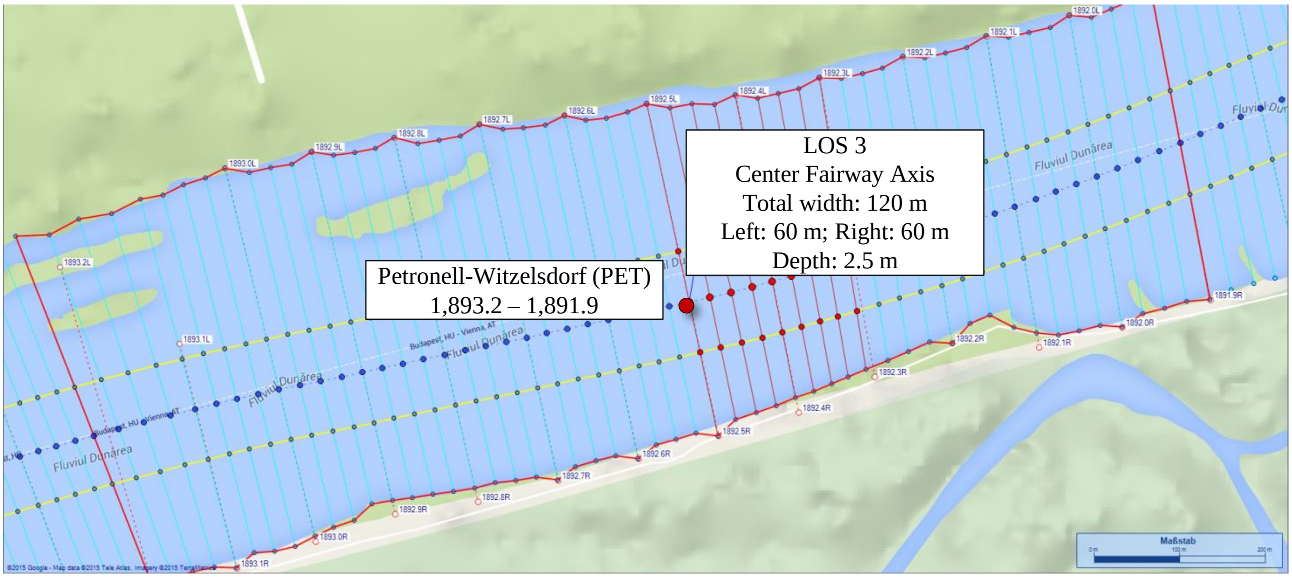
Figure 90: Example of cross-sectional profiles based on their coordinates and their rkm; the intersections with the border lines of the fairway are calculated as well as the mid-point of the fairway together with the horizontal distances to the left and right border lines.

depth of the fairway will be presented in chapter 4.4.4.

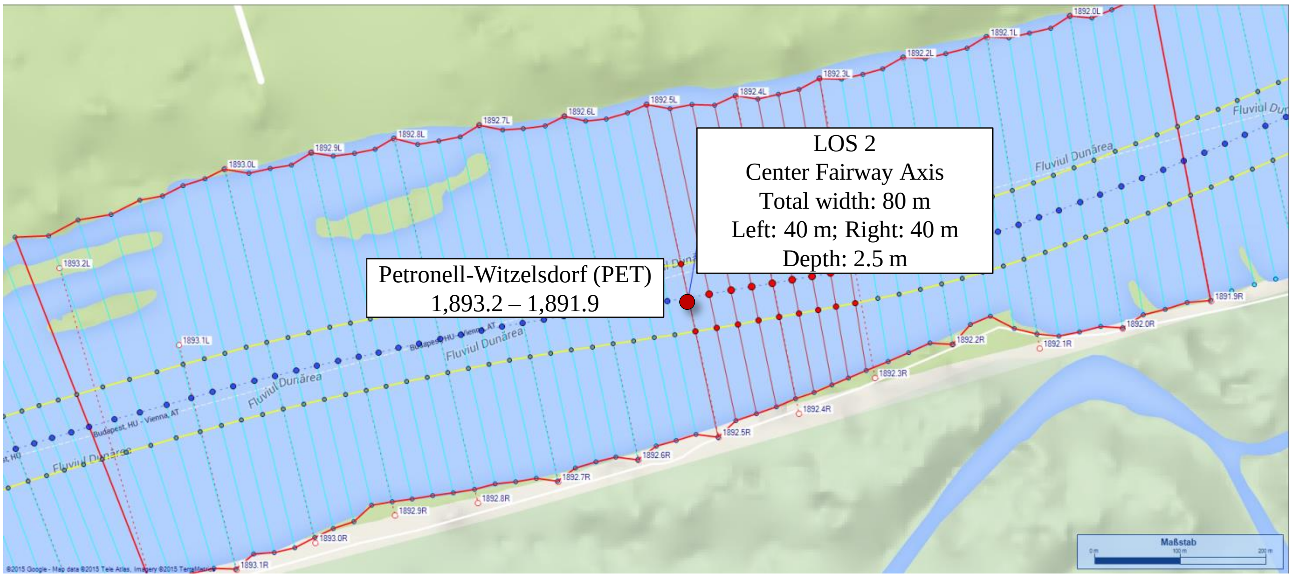
4.4.4 Example WAMS: fairway and levels of service

This chapter provides an overview of the implementation of the three operational levels of service in the WAMS-software, including fairway axis, left and right fairway boundary lines and fairway depth. These are illustrated from a top view in Figure 91 for shallow section PETRONELL-WITZELSDORF. The model of these fairways from a cross-sectional perspective can be found in Figure 93 for the cross-sectional profile at rkm 1892,5 with the top edge of the fairway being linked to the statistical low water level line LNWL. The figure clearly illustrates that it becomes more difficult to identify an area within the cross section with sufficient fairway depths with increasing fairway width. Figure 92 presents the flexible adoption of the fairway channel, exemplified by widening of the fairway channel that can be executed either for individual cross-sectional profiles, single shallow sections or the entire river stretch. The red shaded areas between the fairway bottom and the riverbed already form the introduction for the next chapter of the thesis, the availability concept of the fairway.

WAMS-example: fairway model – Level of Service 3 (width=120.1m, depth=2.5m); rkm 1,892.500



WAMS-example: fairway model – Level of Service 2 (width=80.1m, depth=2.5m); rkm 1,892.500



WAMS-example: fairway model – Level of Service 1 (width=40.1m, depth=2.5m); rkm 1,892.500

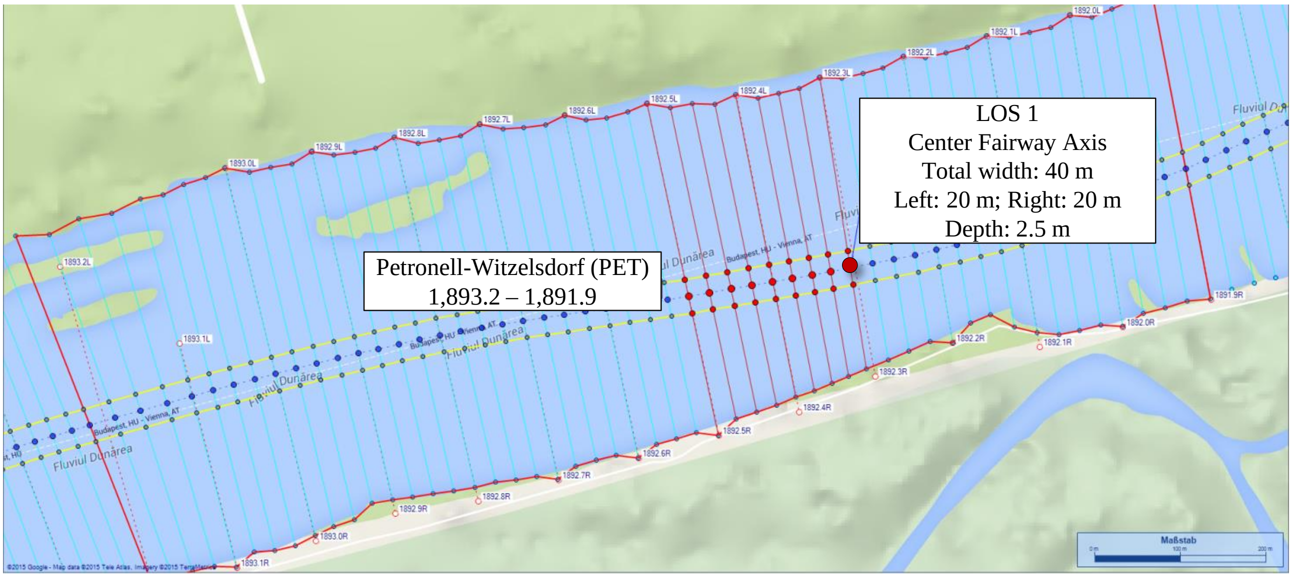


Figure 91: WAMS – fairway model with defined Levels of Service LOS1, LOS2 & LOS3 for shallow section Petronell-Witzelsdorf

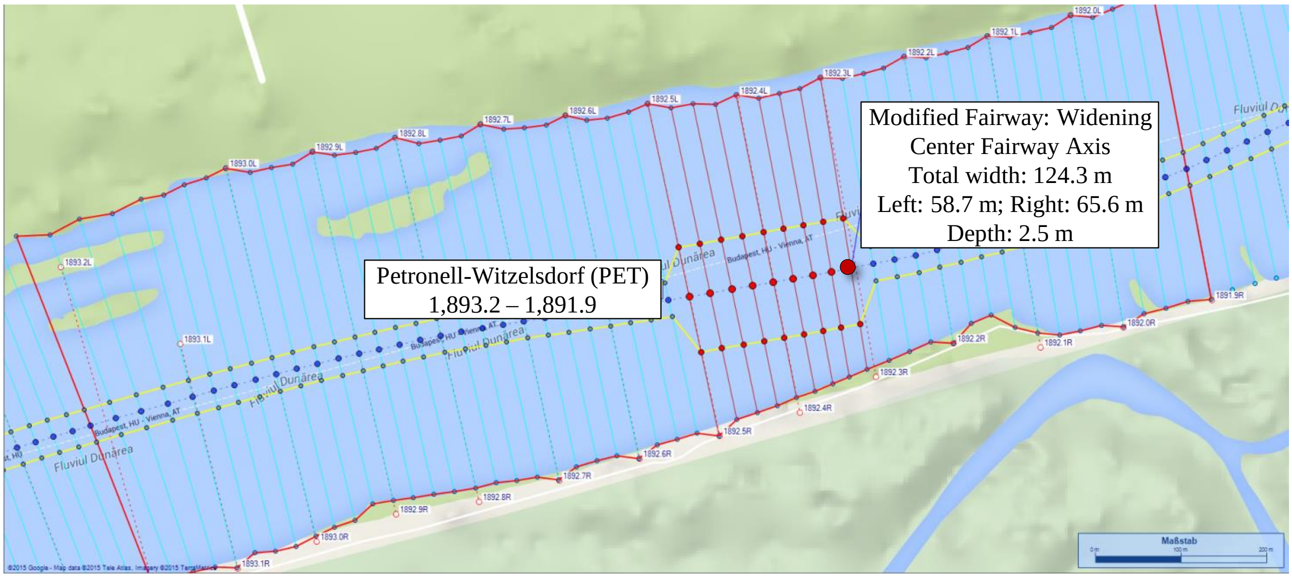


Figure 92: Optional functionality: Flexible adaption of fairway width as important tool for fairway operation allowing fast shifting and narrowing of the navigational channel

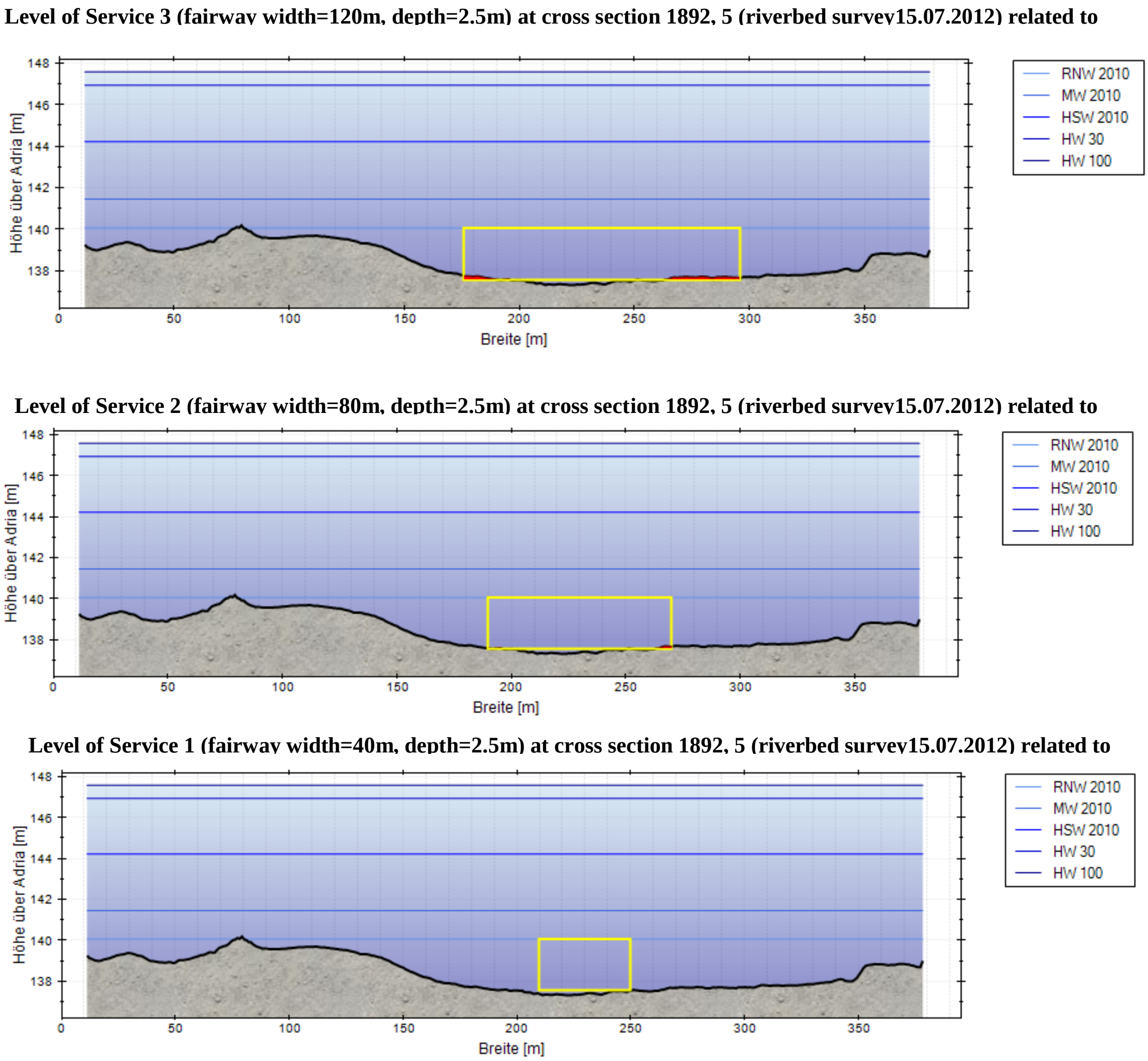


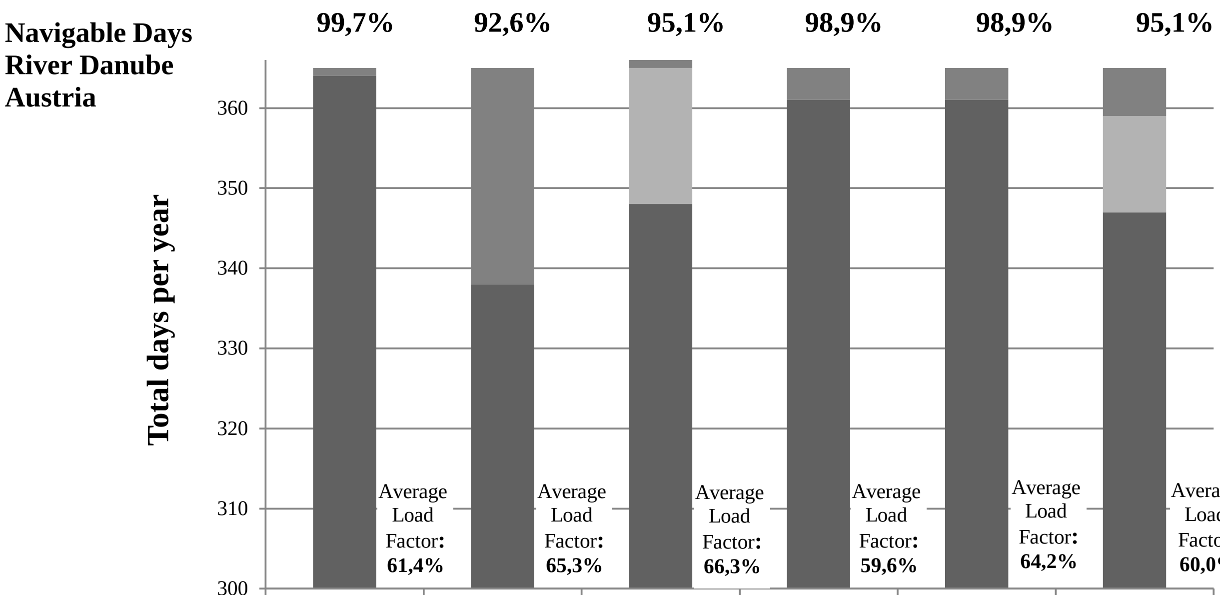
Figure 93: Cross sectional profile presenting different levels of fairway width (LOS3, LOS2, LOS1) – WAMS-Software

4.5 Fairway availability

4.5.1 Current availability definitions and concepts

The availability of waterways has been equated with general navigability of inland waterways in the terminology of international waterway authorities. This means that a full availability is given for waterways that are generally considered to be navigable, regardless of the possible utilization. Thus, waterways are only considered as non-navigable in the case of floods or severe ice formations. Closures of navigation due to high water levels occur mainly in spring (due to snowmelt) or in mid-summer. Closures due to ice formation show, in average, a duration of 16 days and are mainly found in January or February [VIADONAU, 2009-2014].

However, there are no closures of navigation due to low water events. Consequently, there are periods where cargo vessels are able to use the waterway only to a very limited extent, in terms of economic efficiency, as efficiency of inland waterway transport is closely related to vessel utilization. This reduced fairway availability is described as a bad fairway condition. Figure 94 provides an overview of official fairway availability of the Austrian Danube stretch, also including closures due to high water events and ice formation as well as average vessel utilization in the respective year. According to annual reports of the Austrian waterway agency VIADONAU, the availability of the fairway range from 92.6% in 2013 (27 days with closure due to high water levels) up to 99.7% in 2014 (1 day with closure due to high water levels) with average annual load factors ranging between 59.6 % in 2011 and 66.3% in 2012. Naturally, annual utilization values are far higher than the minimum load factors in single months with low water levels, where load factor might fall far below 50 % of the capacity. Figure 95 underlines the correlation between average monthly load factors and water level development, exemplified for gauge Wildungsmauer in 2014. The figure clearly indicates the vessel utilization is decreasing with decreases water levels; significant low utilizations are given when the water level falls below the statistical low navigable water level LNWL.



	2014	2013	2012	2011	2010	2009
Closures due to high water	1	27	1	4	4	6
Closures due to ice	0	0	17	0	0	12
Navigable Days	364	338	348	361	361	347

Figure 94: Current concept of availability with differentiation between navigable days, days with closure due to ice, and days with closure due to high water level providing an overview on resulting fairway availability between 2009 and 2014 reaching 92,6% up to 99,7%. [Annual reports VIADONAU, 2009-2014]

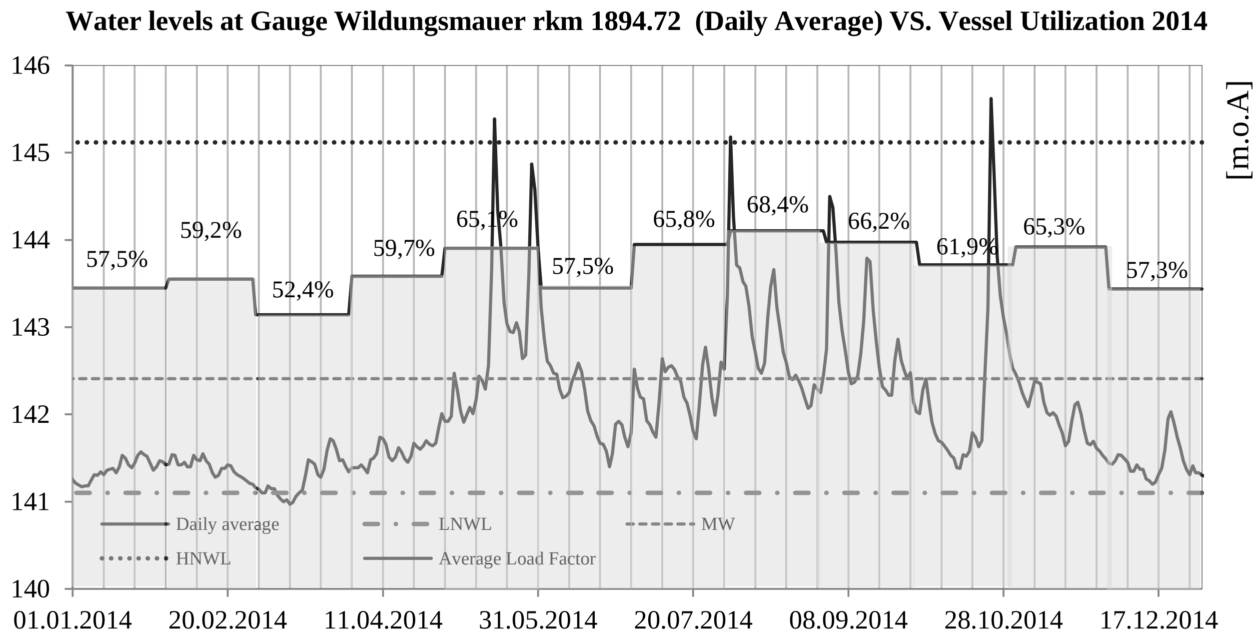


Figure 95: Average monthly vessel load factor 2014 compared with average water level development at reference gauging station Wildungsmauer; annual report [VIADONAU 2014]

Due to the fact that inland waterways have to be considered as serial systems in terms of continuous navigability of the entire waterway, approaches focusing only on local river stretches fall short regarding planning of transport processes, because for the trip of a vessel between two ports, the worst fairway condition is decisive for the utilization of a vessel or convoy. Thus, navigable feeder rivers and connecting channels as the Main-Danube canal linking the Danube with Main and Rhine waterways have to also be included in any availability considerations if they are part of the transport route. The Main-Danube Canal generally shows a lower availability (e.g. 87.9% in 2012) than the Danube waterway. This lower availability is especially important for transport coming from the west. However, if currently available fairway depths are not included in the analysis, there will be no way of determining the necessary extent of maintenance measures. Current definitions of fairway availability as yes/no criteria on a transport route thus fall somewhat short with regard to customer needs and measure optimization.

For inland waterways, a serial transport system as well as power plant and their locks have to be included in a comprehensive availability approach. In general, the Danube locks are equipped with two lock chambers and thus provide a high reliability. Closures due to technical defects, maintenance work and dredging only affected individual chambers and lasted for short periods. According to the Austrian waterway agency VIADONAU, in 91% of all cases, chambers were closed for less than one day and for an average of only 4.5 hours. If only one lock chamber has to be closed, waiting times might appear, but are not very likely due to a low utilization of locks, amounting to only 12% on average. If a failure includes both lock chambers, the fairway has to be considered as non-available until regular operation is possible.

A comprehensive availability concept must represent the competitiveness of the mode of transport, which is mainly determined by the total availability and reliability of the overall system throughout the year. The new comprehensive availability approach (presented in chapter 4.5.2) therefore includes availability of lock-chambers, closures due to ice formations and high water levels, as well as a reduced availability due to low water events.

4.5.2 New availability approach

For navigation companies, the availability of minimum fairway widths and depths is crucial both for planning individual transport trips and for being competitive throughout the year. Based on data from water levels, as well as riverbed surveys and temperature information, it is possible to calculate the availability for any given timeframe, cross section, river section or entire transport route according to equation (51)

$$AV_{d;w} = \sum_{i=1}^{365} t_i - \sum_{i=1}^{365} t_{i_{Ice}} - \sum_{i=1}^{365} t_{i_{HW}} - \sum_{i=1}^{365} t_{i_F} \qquad \textit{for all } \mathrm{d}_i > d, w_i > w \tag{51}$$

with $AV_{d,w}$ = total availability of a fairway class in days per year: t_i = available time in days; t_{iIce} = days with closure due to ice, t_{iHW} = days with closure due to high water levels, t_{iF} = days with closure due to lock failures or accidents, d = fairway depth, w = fairway width.

For any serial system having closures of navigation due to lock failures or vessel accidents, ice formations and high water levels have to be included in availability considerations of a transport route as well as a reduced availability of connecting channels. Thus, the total availability may be different, depending on the considered transport relation. The total availability of fairway width and depth of a respective transport route is consequently reduced by the sum of days with closures due to ice, the sum of days with closures due to high water levels and the sum of days with closures due to accidents or failures. As a result, a closure due to ice formations on the Austrian Danube stretch lasting for 43 days would mean that the availability surface is reduce by an offset value of 43 days, as illustrated in Figure 98. [Haselbauer, K. et al. 2014].

In order to model the availability of the fairway, the top edge of the course of the fairway is linked to the water surface. The altitude of the fairway's top and bottom varies on a daily basis according to the current water level. The physical availability for a river section on a specific day results from the non-intersection of a combination of width and depth of the fairway with the riverbed. Starting with the fairway axis and the minimum fairway dimensions, each combination of fairway depth and width is analyzed (Figure 97). If this procedure is repeated for all days of a period, the availability percentage may be defined as the number of days with non-intersection divided by the total number of days in the analyzed period. For any river section or river stretch, the resulting availability will decrease with increasing fairway dimensions and result in a convexly falling availability performance surface (Figure 96). The gradient of curvature for this surface strongly depends on the geometry of the riverbed and the course of the fairway in the riverbed. An almost plain availability surface is typical for wide shallow sections with a uniform extensive sedimentation along the river axis. Lateral sedimentation implicates that the availability performance of the fairway shows a sudden drop if a specific fairway width is exceeded. [Haselbauer, K. et al. 2014].

Based on this approach, any recommendation may be described as a single point. If this point is below the actual availability surface, then the recommendations are met in the analyzed period. Otherwise, additional physical measures (e.g. dredging) have to be implemented in order to achieve the targeted availability resulting in an improved availability performance that may be described in an upward shift of the availability surface [Haselbauer, K. et al. 2014].

In order to find the decisive availability performance of a transport route, including connecting channels (Main-Danube-Canal) and feeding rivers (Save), their availability performance has to be compared to the performance of all other river stretches of a transport route. The most critical availability performance determines the total availability of the transport route.

Figure 96: Resulting 3D-availability performance including all combinations of fairway width and fairway depth of a specific river section for one year compared to various availability targets (Levels of Service) like for example LOS 3: width = 120 m, depth = 2.5 m, 343 days; the difference in available days between target availability and actual availability has to be compensated by appropriate dredging measures [Haselbauer, K. et al. 2014]

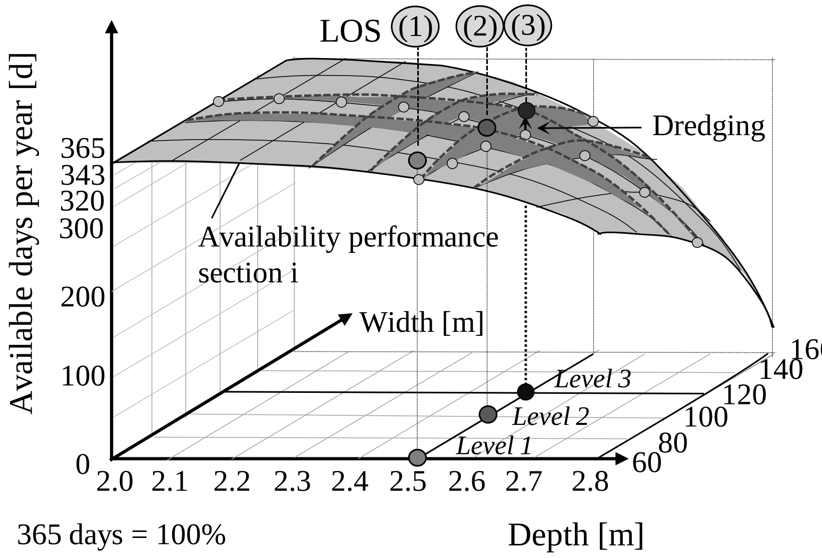


Figure 97: Intersection of classes of fairway width and depth linked to the actual water level with the actual riverbed as basic input parameters for the calculation of a 3D-availability surface exemplified for a specific river section [Haselbauer, K. et al. 2014].

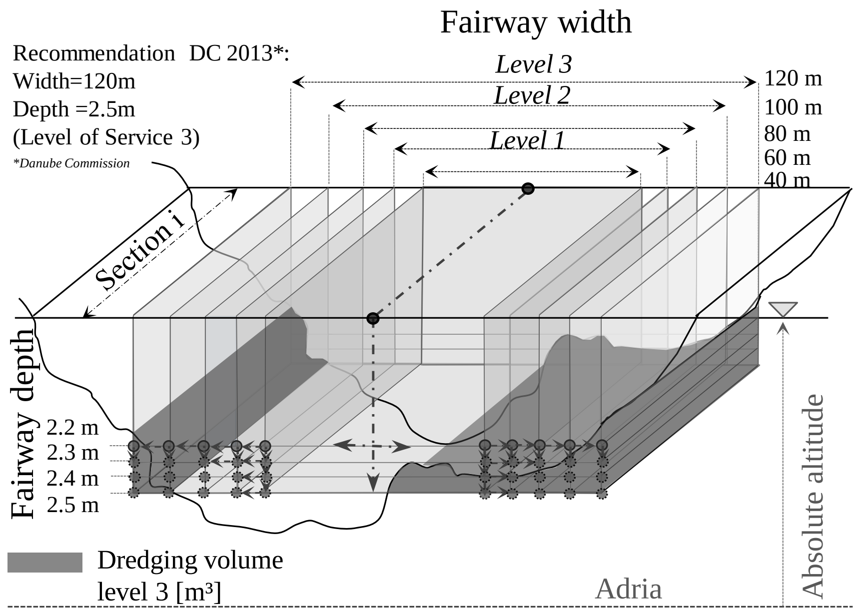
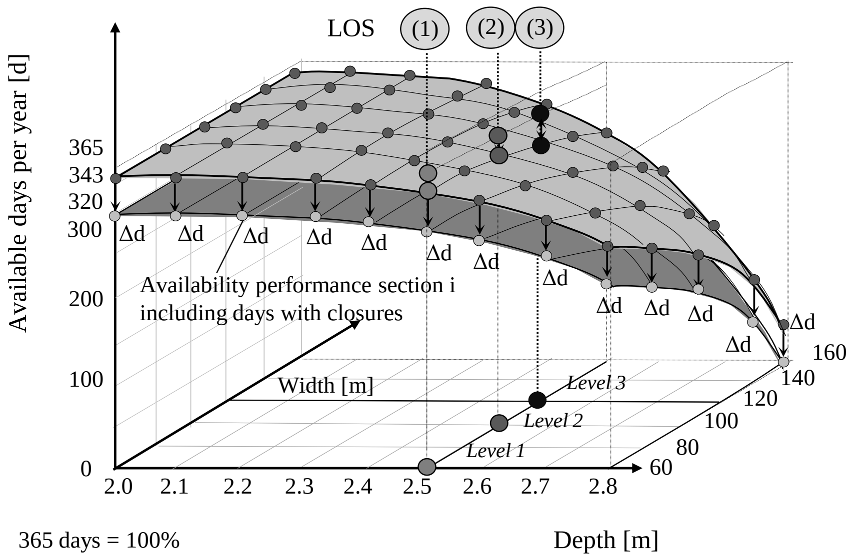


Figure 98: In the case of closures due to ice formations or high water levels the total availability performance has to be reduced by the days of closures for any combination of fairway width and depth. This may be visualized by a parallel shift of the entire availability surface downwards.



4.5.3 Example WAMS: availability calculation on the river Danube

The presented waterway asset management approach is currently being implemented on the entire Austrian section of the Danube waterway. As an example, the availability of one of the most critical shallow sections (ford Schwallenbach in the free-flowing Wachau stretch) was investigated for the year 2011. This year was characterized by particularly unfavorable water levels for inland navigation. The targeted fairway availability on this critical section for LOS 3 was only met on 234 days (64.1%), for LOS 2 on 257 days (70.4%) and for LOS 1 on 269 days (73.7%). Thus, the recommended fairway availability of 343 days (Figure 28: LOS 1: -74 d, LOS 2: -86 d, LOS 3: -109 d) could not be met. This example clearly demonstrates that the target availability of 94% of days per year could have only been achieved by implementing further physical measures to a substantial extent.

In order to get a more accurate picture of the availability performance during the year 2011, the development of fairway availability was evaluated on a monthly basis. The availability surface for each month is illustrated in Figure 100 together with the availability of recommended fairway parameters (LOS 3: width = 120m, depth = 2.5 m). The analysis shows excellent fairway availability during the summer months of June, July and August with fairway LOS 3 being available during 90 to 100% of the month. The analysis also indicates a particularly poor availability performance in typical low-water periods during the months of September and November. With a fairway availability of only 6% for the target parameters, the month of November in 2011 had severe negative consequences on waterway transport. Overall, inland navigation could use only a fairway depth of 1.9 m throughout the entire month based on the analysis of this critical sector. For a smaller fairway width, a depth of 2.2 m was available on at least 50% of days in November. The assessment of availability on a monthly basis allows a better understanding of the river and the specific characteristics of critical sections. This may, in turn, lead to an improved planning of future transport operations, especially in goods transport, as well as the timing of river maintenance and engineering measures. For availability prediction, historical discharge data will enable the calculation of probability levels for different target availability performances, which is based on previous water level distribution [Haselbauer, K. et al. 2014].

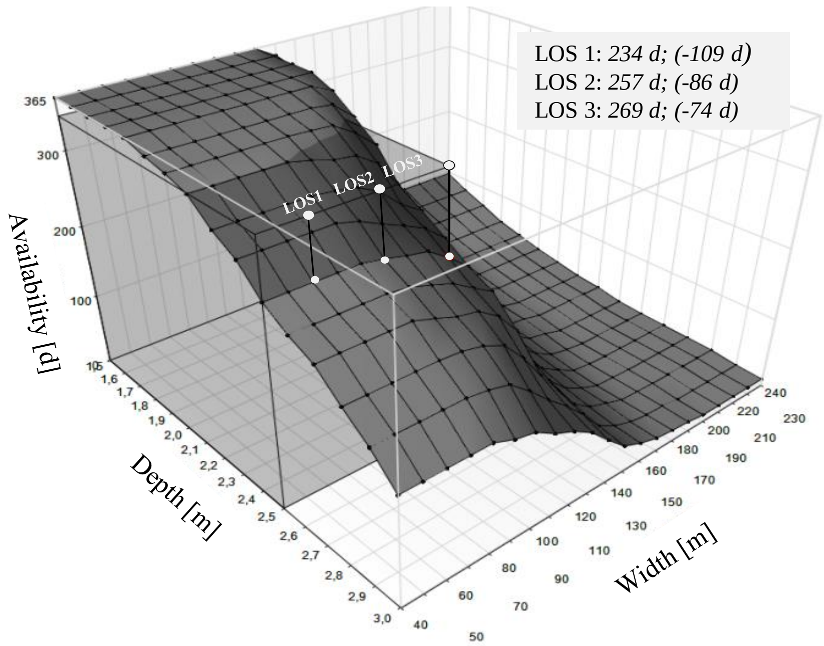


Figure 99: Availability performance of ford Schwallenbach (rkm 2021.975 – 2022.5) for the year 2011 including different service levels (LOS 1, LOS 2, LOS 3) [Haselbauer, K. et al. 2014].

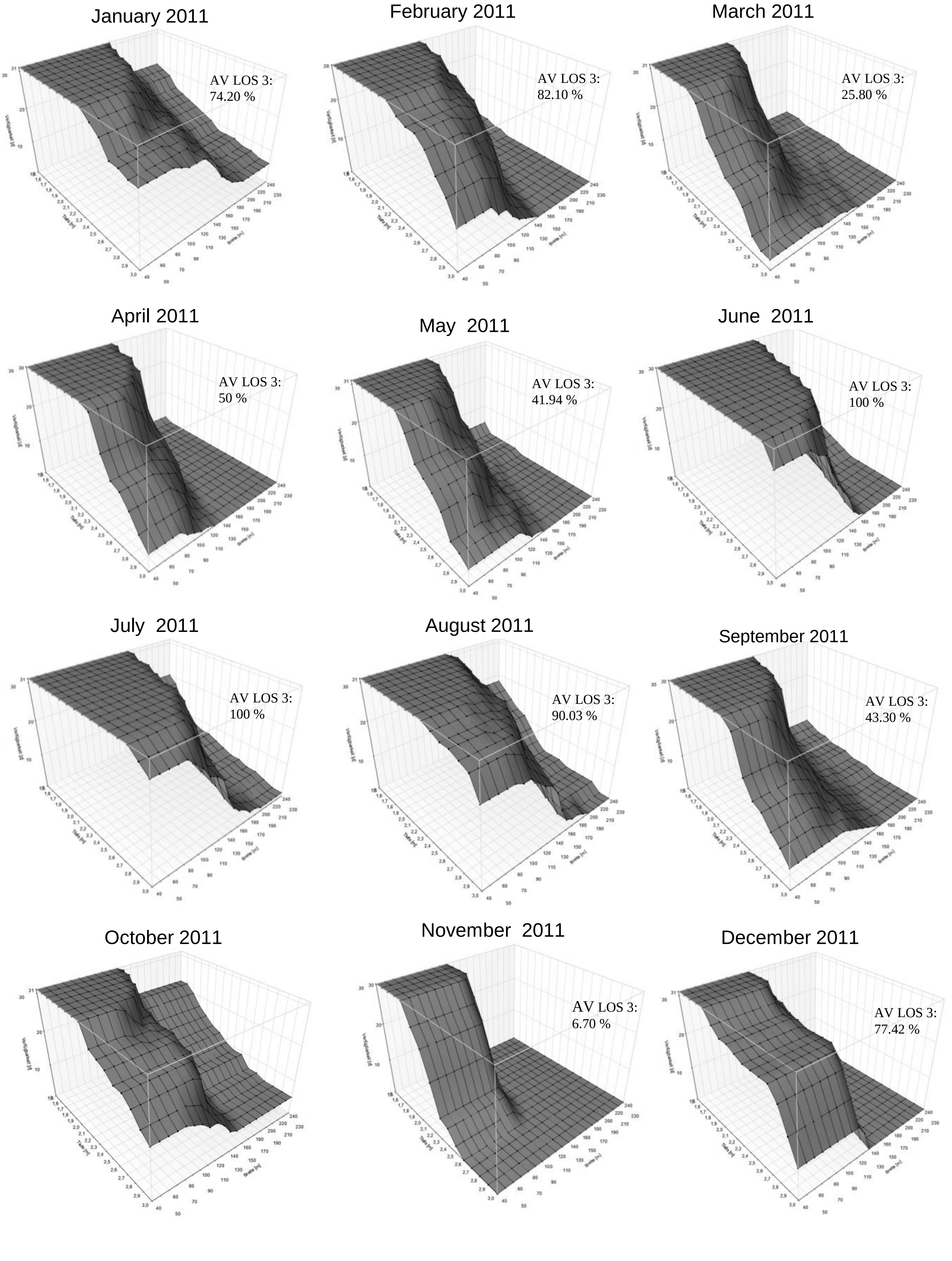


Figure 100: Ford Schwallenbach - development of the availability performance on a monthly basis during the year 2011 (rkm 2021.975 – 2022.5) [Haselbauer, K. et al. 2014].

4.6 Prediction of fairway conditions

In waterway asset management, the predictions of condition parameters of the fairway represent a challenging task as rivers show very dynamic performance behavior and are affected by a considerable number of influencing parameters, which requires many research disciplines being engaged. Nevertheless, based on a robust amount of basic data, a forecast of water levels, riverbed development and failures of import system components, such as locks, can be derived by taking their occurrence probabilities into account and thus provide basic information for planning of transport processes. For this purpose, the comprehensive availability model is divided into the basic system components: water level, riverbed, and fairway class, closures due to ice, floods and accidents.

Failure probabilities of individual lock-facilities can be calculated based on long-term data sets. The same applies for other possible causes for navigational closures such as vessel accidents, ice formations or floods. For a certain transport route, the resulting days of non-availability are summed up for the estimation of the total availability performance.

Condition prediction in waterway asset management is generally based on a probabilistic empirical approach, where the distribution of water levels and the distribution of sedimentation and erosion processes for individual shallow sections are determined by a statistical analysis of long-term data sets. Waterway asset management aims at a prediction of fairway conditions, with and without measures, in order to subsequently compare the resulting availability performances. The prediction of the impact of individual measures on fairway availability and the duration of the impact are of significant importance, especially for the economic comparison of measure alternatives. In order to allow such predictions, a reliable data base including the measure impact on availability and the impact duration is required to subsequently derive the necessary statistical distributions. Furthermore, for the prediction of fairway conditions as the core of the new availability approach is based on an empirical analysis of long-term data series of water level development, riverbed development and sediment measurements of individual shallow sections. The prediction of water levels is based on extrapolation of water level and discharge data, measured during a period exceeding 30 years. As a result, water level forecasts can be calculated on a monthly basis, including occurrence probability and deviation.

Since individual shallow sections show significantly varying behavior regarding sedimentation and erosion processes, predictions have to focus on single characteristical sections. Inherently, the reliability of these forecasts increases with increasing amounts of basic data and shorter forecast periods. The prediction of riverbed development is considered as most complex, as several impact parameters have to be considered. Therefore, a self-learning system for the condition development of individual shallow sections is proposed. The reliability of the condition prediction increases with an increasing number of riverbed surveys that are linked to the discharge development. Based on the approach, a probability of certain riverbed conditions may be derived for any shallow section. If measures have been implemented at a shallow section, the backfilling behavior of the dredged volume has to be derived as well, based on a number of consecutive riverbed surveys. This backfilling behavior may be analyzed for all shallow sections, linked to all dredging measures, and be included in the waterway asset management database. In the case of the implementation of dredging measures, the riverbed altitude of a certain cross-sectional point is reduced by a specific offset value as shown in Figure 101. Generally, a reduction of the water level altitude by the same reset value would be required. However, when only a single cross-sectional point or cross section is considered, this effect is negligible. By contrast, dredging measures that involve longer river sections, and the entire fairway width, will result in a reduction of the water level altitude as well, which may be detected empirically in WAMS.

If an arbitrary point of a cross sectional profile is selected and the development of water level and riverbed for this point are displayed as hydrographs over time, the availability of different fairway depths may easily be identified and extrapolated, providing a certain probability of fairway availability for predictions as well for long-term data sets. Figure 102 illustrates a perennial hydrograph of water level, fairway classes and riverbed for a cross-sectional point. Different levels of fairway depth are illustrated as parallel lines to the hydrograph of the water level. The shaded areas between the riverbed development and the bottom lines of different fairway depths mark days with non-availability of the respective fairway depth. This illustration of a two-dimensional model of fairway availability over time may also be extended to a four-dimensional model, which is realized by as a basic module of the WAMS software. As afore mentioned, the prediction of the riverbed development must be based on several riverbed surveys, discharge data and sediment measurements, which have to be evaluated for the entire Danube stretch. An appropriate functionality for these evaluations based on the presented methodical approach will be implemented in the WAMS software tool to allow forecasts of fairway availability with and without measures.

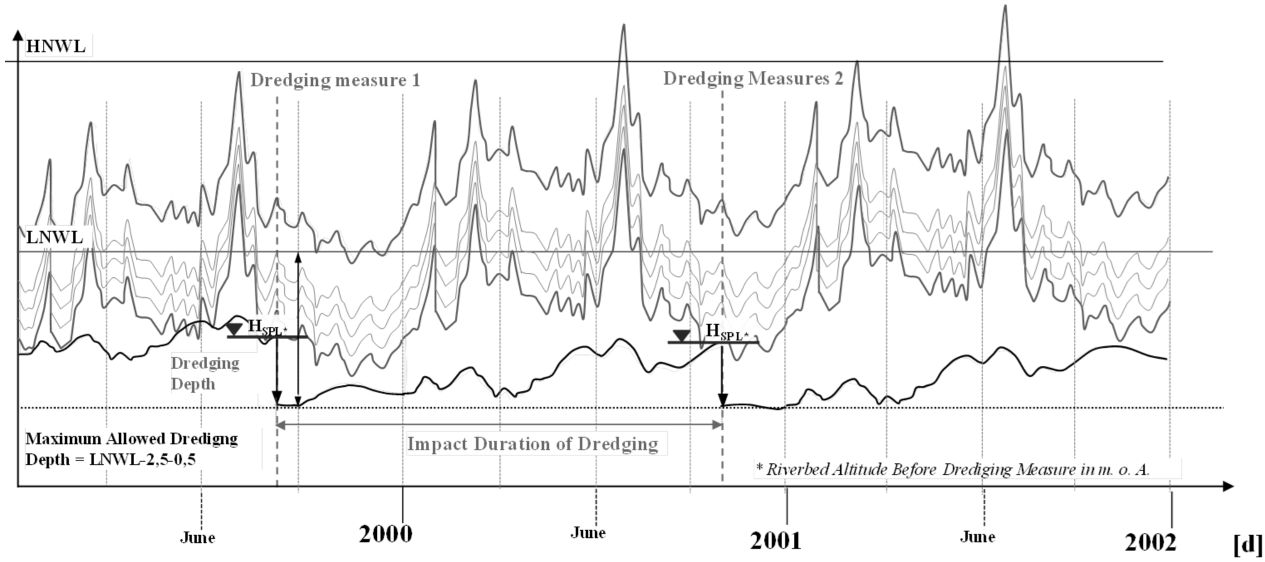


Figure 101: Development of riverbed, water level and fairway conditions with dredging measures

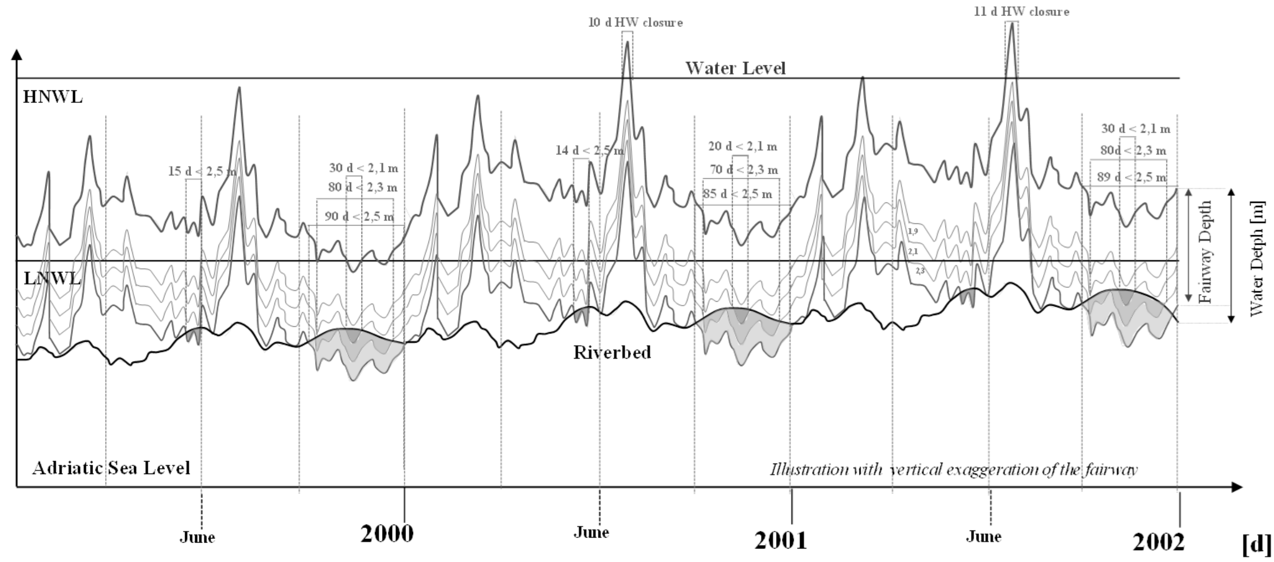


Figure 102: Development of riverbed, water level and fairway conditions without measures

5 MEASURES IN WATERWAY ASSET MANAGMENT

In order to increase fairway availability, waterway authorities may choose between various possible measures with different costs, impact on availability, realization time, duration of impact, resulting user costs and environmental impact. For the purposes of waterway asset management approaches that mainly focus on fairway availability, three main categories of measures are generally applied. These main categories are operational measures (narrowing, widening and shifting of the fairway), maintenance measures (dredging) and river engineering measures (construction of groins), and will be further described in the following chapters 5.2 and 5.3. Depending on urgency, environmental restrictions and available budgetary resources, agencies have to select the most appropriate measure in terms of fairway availability and efficiency. The main goal of waterway operators is to provide the best possible and uniform level of service throughout the entire year to their customers while being as efficient as possible within the framework of their available budgetary resources. For this purpose, the impacts of measures on the waterway model, the fairway availability, the duration of measures impact and their costs must be known. Determining optimal measures on individual sections always follows the ruling principle of targeted fairway availability among other conditions and constraints. Based on these circumstances all possible measures and the zero alternative are evaluated leading to a measure program containing the best measures for all shallow sections. The input parameters for the economic comparison of different measure types are presented in the chapter of the respective measure. Despite minimum operator costs, an optimization toward minimum costs for other stakeholders, such as customers or the environment is also possible. However, as available budget is limited, this thesis focuses on how to invest limited funds in a way that best suits the needs of waterway users.

5.1 Overview of measures for improving fairway availability

The following section will provide an overview of main measures in waterway asset management and their impact on fairway availability exemplified for a shallow section with a given availability performance (zero alternative) for a fixed fairway depth of 2.5 m and fairway widths varying between 40 and 160 m, as well as a given availability target (DC 2013, 343 days with availability) as visualized in Figure 103.

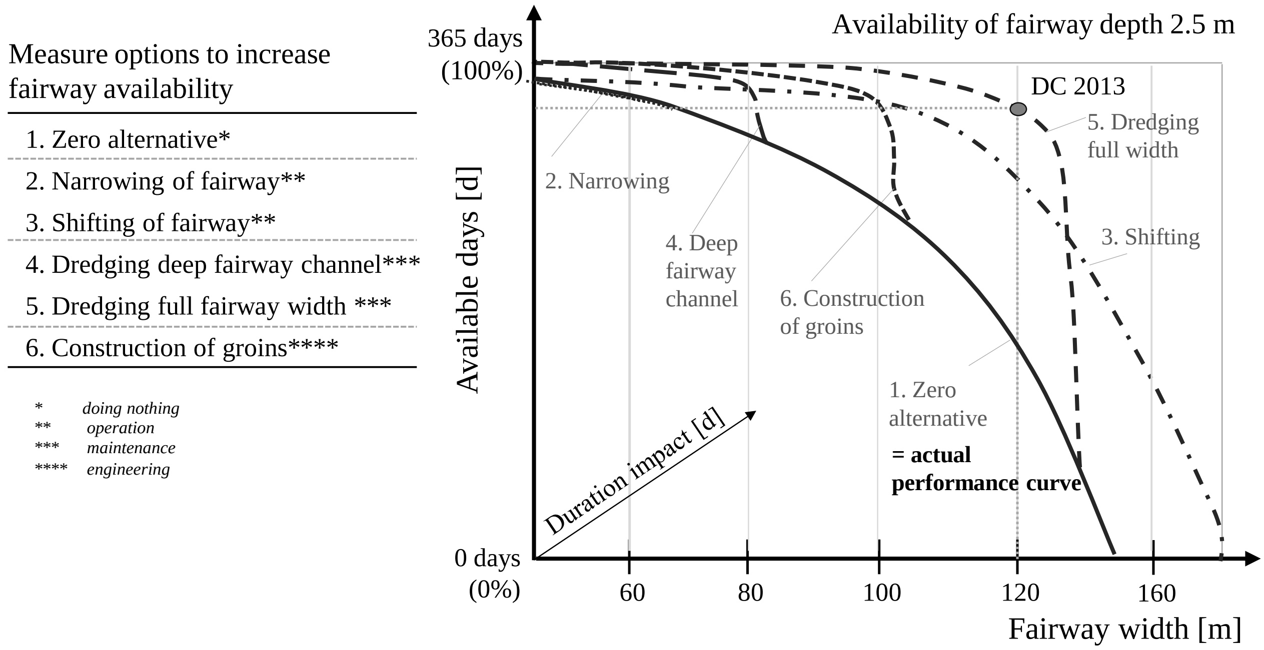


Figure 103: Schematic illustration of actual availability performance of a shallow section and visualization of measure impact of operation, maintenance and engineering measures aiming to achieve the given availability target.

The performance curve (named zero alternative) visualized in Figure 103 may be described as a sectional view of the 3D availability surface presented in Figure 96. The given availability target of 343 days, which should be provided for a fairway, with a depth of 2.5m and a width of 120m is not reached by far. Without the implementation of physical measures, the target fairway depth of 2.5 meters can only be provided on a fairway width of 60 meters. If the fairway depth of 2.5 m has to be provided for the availability target for a fairway width exceeding 60 meters, several measures visualized in Figure 103 with a different impact on availability may be implemented.

In order to find the optimal measure for one shallow section all measures have to be compared to the current situation (“status quo” or “doing nothing”) and to each other. The measure with the highest impact on availability compared to annual costs is considered as favorable. The basic assumptions used to determine the respective annuities are presented in the chapters describing individual measures.

Operational measures, i.e. narrowing or shifting of the fairway, may be applied in order to improve the utilization of the available fairway only in such cases where the target fairway depth is available on a sufficient number of days at least in one adequately wide area of the cross-sectional profile. If the recommended fairway depth is not available for the entire fairway width, narrowing the fairway to those areas with sufficient water depths together with appropriate marking will allow a better utilization of availability. As traffic volumes are quite low compared to the capacity, waiting times due to one lane traffic at certain river sections will not lead to any substantial increase in waiting times. The effort for measure implementation is rather low as well as the implementation duration. Operational measures are generally not assumed to have severe negative impacts on the environment.

For typical wider river sections on the lower Danube showing a higher physical availability outside of the current fairway, shifting of the course of the fairway may also be a cost-efficient and long lasting option. A certain stability of the riverbed is required for a permanent relocation. This measure is often used in the case of a vessel accident to further enable transport operations. However, a successful implementation of operational measures requires periodic riverbed surveys, data processing and information of customers with an interval of at least two to four weeks in critical low-water periods and river sections.

For river sections without a sufficient width and depth, only physical, i.e. maintenance measures, may lead to an increase in fairway availability. The least costly measure would be dredging a “deep fairway channel” on a minimum necessary width (LOS 1) in order to provide a continuous availability of a targeted fairway depth. On river sections with very high transport volumes and no budgetary or environmental restrictions, dredging the entire fairway width according to international recommendations may be considered as favorable. However, on river stretches which are characterized by very dynamic river morphology, the duration of dredging impact may be insufficient leading to the question of more sustainable measures. Such measures may be described as river engineering measures and include the construction of new, and/or adaptation of, existing training structures such as, e.g., groins, training walls or bottom sills. Typically, the planning and implementation of this kind of measures takes longer and they are more costly as well. First evaluations show that measures such as the dredging of a "deep fairway channel" or the narrowing of the fairway especially lead to favorable improvements in terms of navigability with a positive impact on transport costs.

In summary, a careful assessment of the individual situation is always necessary due to different types of river morphology and resulting deviations in the costs and impacts of possible measures. For a successful planning and implementation of possible measures, periodic riverbed surveys are mandatory along with an assessment of the actual impact and costs of already implemented measures.

5.2 Operational measures

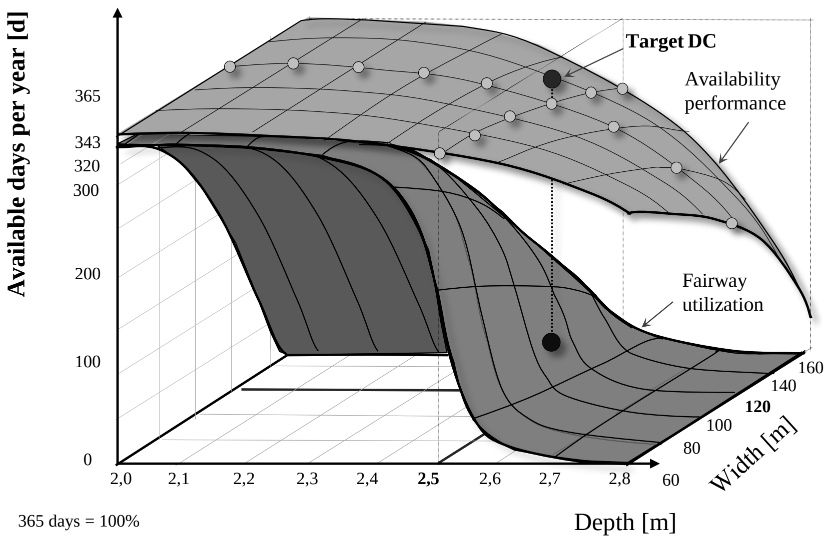
5.2.1 Utilization of available fairway parameters

Operational measures belong to the main tasks in waterway management and involve monitoring of available fairway depths, adaption of the fairway path, marking of shallow sections with buoys as well as the operation of river information systems. In order to support planning, execution and evaluation of operational measures by a software solution, a number of requirements have to be met by such a WAMS-tool. These include a flexible adaption of the fairway channel, subsequently followed by a fast execution of marking and appropriate communication to customers. Furthermore, the visualization and adaption of buoy position as well as the inventory of all other important navigational marks. This chapter will provide, at the beginning, a description of the importance of operational measures in waterway asset management, followed by the practical implementation of operational measures, including working steps and data processing. Moreover, the basic estimations for cost calculations of fairway operation will be presented. For the operation of inland waterways, a number of excellent information systems, such as the river information system (RIS), are already available, which will not be further described in this thesis. However, possible additional features for these systems, in the context of waterway asset management, will be introduced.

Operational measures due not induce any physical changes of riverbed geometry or water level altitude and thus do not affect fairway availability directly. However, these measures are highly suitable to communicate to customers which fairway depth is available and, thus, support planning of transports and increase the safety during the transport process as well through appropriate marking and signalization. Hence, this measure intends to make physically existing availability more visible and thereby increase the utilization of the fairway.

The actual utilization of physically provided fairway availability by the vessel fleet is a function of various impact parameters such as traffic volume, the accuracy of fairway information or the reliability of water level forecasts and riverbed surveys. These factors, together with the empirical experience of navigation companies, lead to an implicit safety margin between physically possible draughts loaded of vessels and actually used draughts, which can also be characterized as a "trust margin". Therefore, the utilization of available transport capacity over the course of the year will always be below availability performance. Figure 104 provides a principal insight into the ratio of used fairway availability compared to provided fairway widths and depths. Operational measures, i.e. narrowing or shifting of the fairway, generally reduce the gap between utilized and provided infrastructure availability.

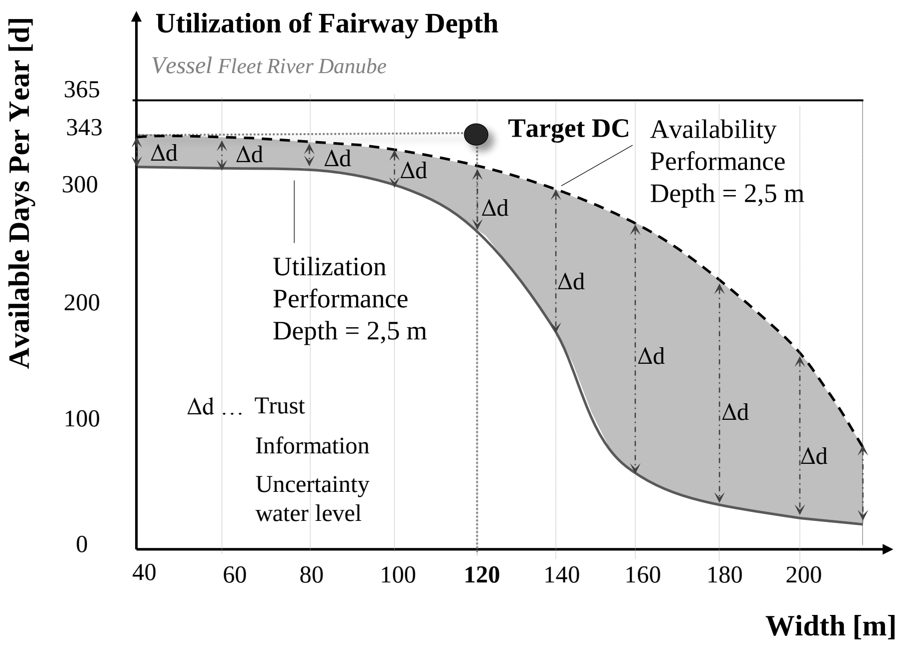
Figure 104: Availability performance of a river section compared to the target availability and the actual utilization of fairway widths due to the existing fleet and current traffic volume [Haselbauer, K. et al. 2014]



The comparison of both, the provided and actually used infrastructure quality is an important indication for infrastructure operators as to whether available infrastructure quality and fairway information meet the demand of the transport market and whether any improvement in a certain direction (e.g. width or depth) will lead to lower transport costs and a higher competitivety of waterway transport. The calculation and visualization of the utilization of fairway depth in a waterway asset management system may be based on a combination of section-related vessel trajectories and data on individual draught loaded, which may be available, e.g., in a (transponder) database. Furthermore, anonymous utilization indicators of navigation companies may be used for more general backward-related evaluations. Figure 105 provides a schematic visualization of actual availability performance for a fixed fairway depth of 2.5 meters for different fairway widths in comparison with utilized fairway depth during the year for different levels of service. For increasing fairway widths, the probability that peripheral areas of the fairway are fully utilized (i.e. with a maximum loaded draught) decreases. For areas in the vicinity of the fairway boundaries, the frequency of vessels passing is generally lower because the level of uncertainty increases notional in terms of concerns regarding accuracy and actuality of information on fairway depths.

A key parameter for improving utilization of available fairway depths is the accuracy of water level forecasts as a basis for transport planning, with typical transport durations from 1 to 3 weeks. The necessary basic information is currently provided on a patchwork of individual national websites, if at all. However, there are certain projects on the Danube underway aiming at a further improvement and harmonization of information access on one single online platform [Hoffmann M., Haselbauer K. Blab R. 2014]. Nevertheless, in order to ensure a higher utilization of the vessel fleet, information on available fairway depths must be available Danube-wide, and be provided on a common quality level and include all riparian countries. The utilization of fairway widths is mainly a function of traffic volume on transport routes and fleet composition on river sections, and can be clustered based on encounter cases and overtaking maneuvers (compare Figure 105). Figure 106 provides an overview of the utilization concept of fairway width, where the actual availability performance is compared to utilized fairway performances clustered for level of service. The figure indicates that the level of service 1 shows the best utilization of physically given fairway availability. Current low traffic volumes on the upper Danube with an encounter probability of only 4.3% for two convoys with critical dimensions (pushed convoy with four lighters) – which currently only have a share of 4.2% of the fleet operating on the upper Danube – on narrow sections implies that resulting waiting times and costs are almost negligible.

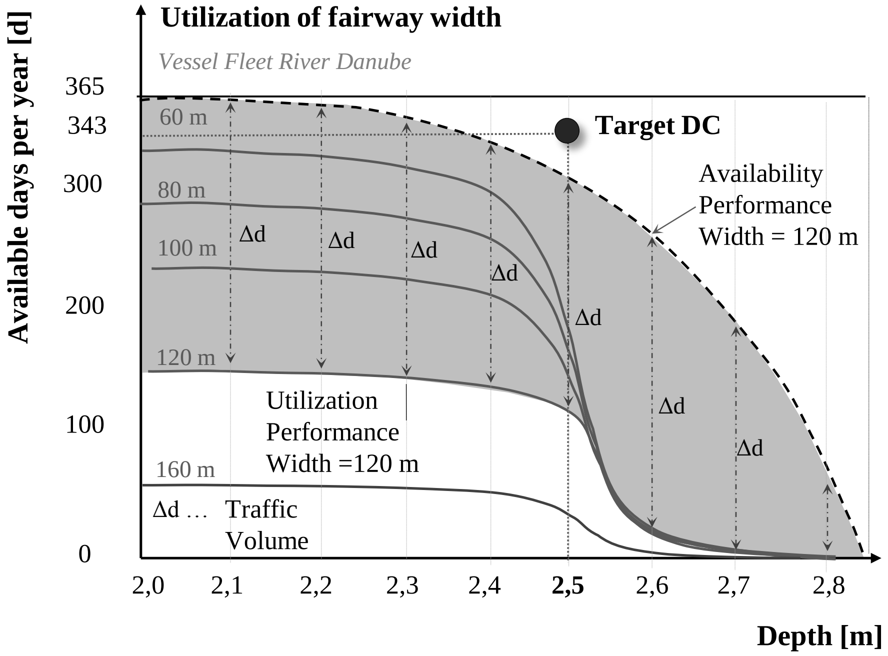
Figure 105: Actual utilization of provided fairway depths for the existing Danube vessel fleet depending on distribution of utilization and encounter probability of different vessel types [Haselbauer, K. et al. 2014]



Therefore, individual vessels or convoys will leave the main traffic lane in the fairway only in the case of an encounter or overtaking maneuver. The actual utilization of fairway widths can be derived on the basis of vessel trajectories, which may be stored in a (transponder) database. A comparison between actual fairway availability and utilized fairway widths could provide the basic data required for an economic assessment and cost-benefit analysis. A comprehensive analysis of vessel trajectories aiming at a determination of the utilization of respective river sections is generally planned, but not yet implemented.

Operational measures, such as narrowing of the fairway (including the appropriate marking of the limits of the fairway), and coordination between approaching vessels on a limited number of river sections may lead to a higher efficiency of inland navigation in general, without the need for substantial additional investments.

Figure 106: Actual utilization of provided fairway widths and depths for the existing Danube vessel fleet depending on distribution of utilization and encounter probability of different vessel configurations [Haselbauer, K. et al. 2014]



5.2.2 Planning, execution and evaluation of operational measures

The marking and alignment of fairway signs is the most common measure to ensure safe passage and improve fairway utilization Danube-wide. Operational measures in a simplified process are applied very often in everyday work of most waterway operators, although, in some cases, the efficiency of planning and implementation could be further improved. In the common terminology of waterway operators, operational measures in waterway asset management are partly covered by the terms signaling and marking. A comprehensive approach of fairway operation, as presented in this thesis, would comprise an evaluation of vessel trajectories in addition to monitoring of the riverbed and buoy position and, therefore, demand multiple interfaces in order to widespread river information systems. Through the use of a WAMS software tool where planning of fairway adaptions may be directly edited from a central workstation, followed by automated relocation of buoys equipped with GPS and automated data transmission, an almost real-time implementation of operational measures would become feasible. Together with an automated publishing of updated navigational charts, quick responses to changing water levels would allow for innovative control solutions for waterway traffic.

According to the UNECE "Guidelines for Waterway Signs and Marking" [UNECE 2013], waterway marking comprises of signs used to regulate navigation on the waterway and floating as well as onshore signs/signals marking the limits of the fairway and navigational hazards. In order to increase traffic safety, kilometer and hectometer markings should be placed wherever possible. The number of floating and onshore marks, as well as any signal and plan for their location, depends on the characteristics of the waterway with the main goal of ensuring navigational safety.

Placement of the marks shall be based on regular surveys of the riverbed together with measures of depth and width of the fairway so that they indicate fairway dimensions. Furthermore, the location and number of all signs and marks has to be laid out in actual plans with the responsible authority being in charge of the right positioning and uninterrupted operation. Moreover, all boat masters have to be informed of the date of installation or removal or any other alterations along with the rules in restricted sections where meeting and passing are prohibited. Figure 107 and Figure 108 provide an overview of typical marking plans with buoys for inland navigation. The number of necessary signs and fairway markings is strongly related to the characteristic of the respective river section. Based on the above-mentioned "Guidelines for Waterway Signs and Marking", Figure 110 provides an overview on all main typical situations [Hoffmann, M. Haselbauer, K. Blab, R. Hartl, T. 2014].

In countries on the central and lower Danube where fairway marking is carried out as a predominate measure; marking of the fairway is usually based on a marking plan created on at least an annual basis (Figure 108 and Figure 107). Thereby, the location of the signals is defined as well as the number and type of signals. In a further step, the signs and their positions are integrated into the electronic navigational charts and published for the users of the waterway. After anchorage of buoys, the inspection and revision of their positions and conditions defining the limits of the fairway dominate the daily work (Figure 109).

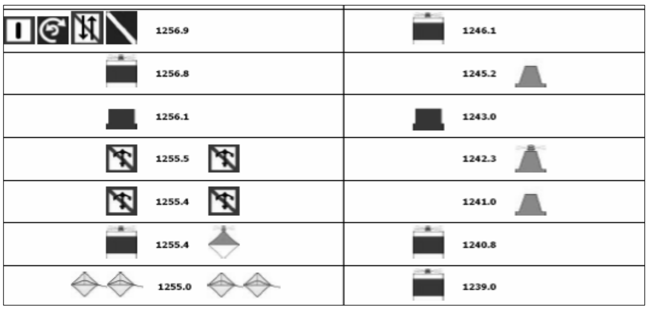


Figure 107: Marking list example including all signs and rkm for a section of the river Danube in Serbia [Hoffmann, M., Haselbauer, K. Blab, R. Hart, T. 2014].

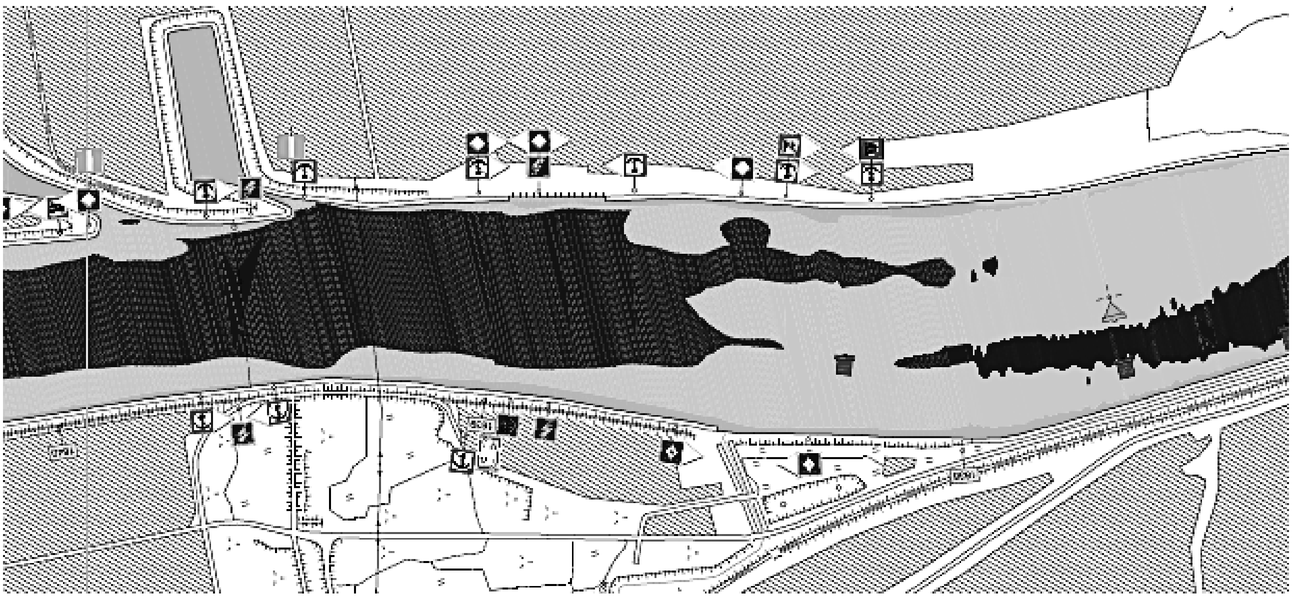


Figure 108: Example for a marking plan visualizing a section of the river Danube in Hungary [Hoffmann, M. Haselbauer, K. Blab, R. Hartl, T. 2014]

Figure 109: Marking activities in practice: example of monitoring of buoy location and condition control [Hoffmann, M. Haselbauer, K. Blab, R. Hartl, T. 2014]

In Austria, the fairway on the Danube is checked between twice a week to twice a month based on three longitudinal single-beam profiles of the riverbed together with the location of all signs. These inspections are carried out with specialized marking vessels and additionally allow for monitoring of the fairway depths along the vessel trajectory by using an integrated echo sounder. These randomly measured fairway depths also serve as an indicator for whether changes in riverbed call for additional detailed surveys (single- or multi-beam) with subsequent adjustment of buoys [Hoffmann, M. Haselbauer, K. Blab, R. Hartl, T. 2014].

However, on this basis, multi-dimensional developments of the riverbed, such as sedimentation processes, cannot be modelled, and fairway availability calculations based on echo sounder data will not be conclusive. The resulting database can be used for random checks on fairway depth, however, these data must be considered as insufficient for all other tasks and objectives of waterway asset management, and particularly also for a modern fairway marking process. In contrast to current practice, with one or two marking plans per year, a dynamic marking approach in a future WAMS would allow for continuous adjustments and monitoring of GPS buoys, depending on actual water levels and navigation requirements, with reduced efforts.

All marking information including possible changes, e.g. due to a narrowing of the fairway on critical sections, are not only part of marking plans and information on websites, but are also published in a standardized form as Inland Electronic Navigational Charts (IENCs). These charts contain all information for safe navigation and are compatible with the standardized Electronic Chart Display and Information Systems (ECDIS). In general, these IENCs are updated once per year with important changes being published on websites, as through notices to skippers or via apps for smartphones as a push service.

As signing is mandatory, there are generally enough signs and buoys available in most waterway agencies as well as a certain number of marking vessels. According to the findings of the field trip and surveys carried out within the project NEWADA duo, the vessel fleet for marking is overaged, leading to certain replacement needs in some agencies. Furthermore, replacing old static buoys on critical sections with new GPS tracked buoys would enable tracking any changes in position and reduce marking/controlling efforts. For most of the river stretches of the Danube, fairway marking can only be recommended for river sections with sufficient fairway depths (> 2.5 m), at least for a reduced fairway width of 40 or 60 meters. Otherwise, physical measures, such as maintenance or engineering measures, are needed in order to achieve the predefined fairway depth. Typically, buoys are equipped with radar reflectors to improve navigability. From a nautical point of view (visibility) the distances between buoys are considered as insufficient [Hoffmann, M. Haselbauer, K. Blab, R. Hartl, T. 2014].

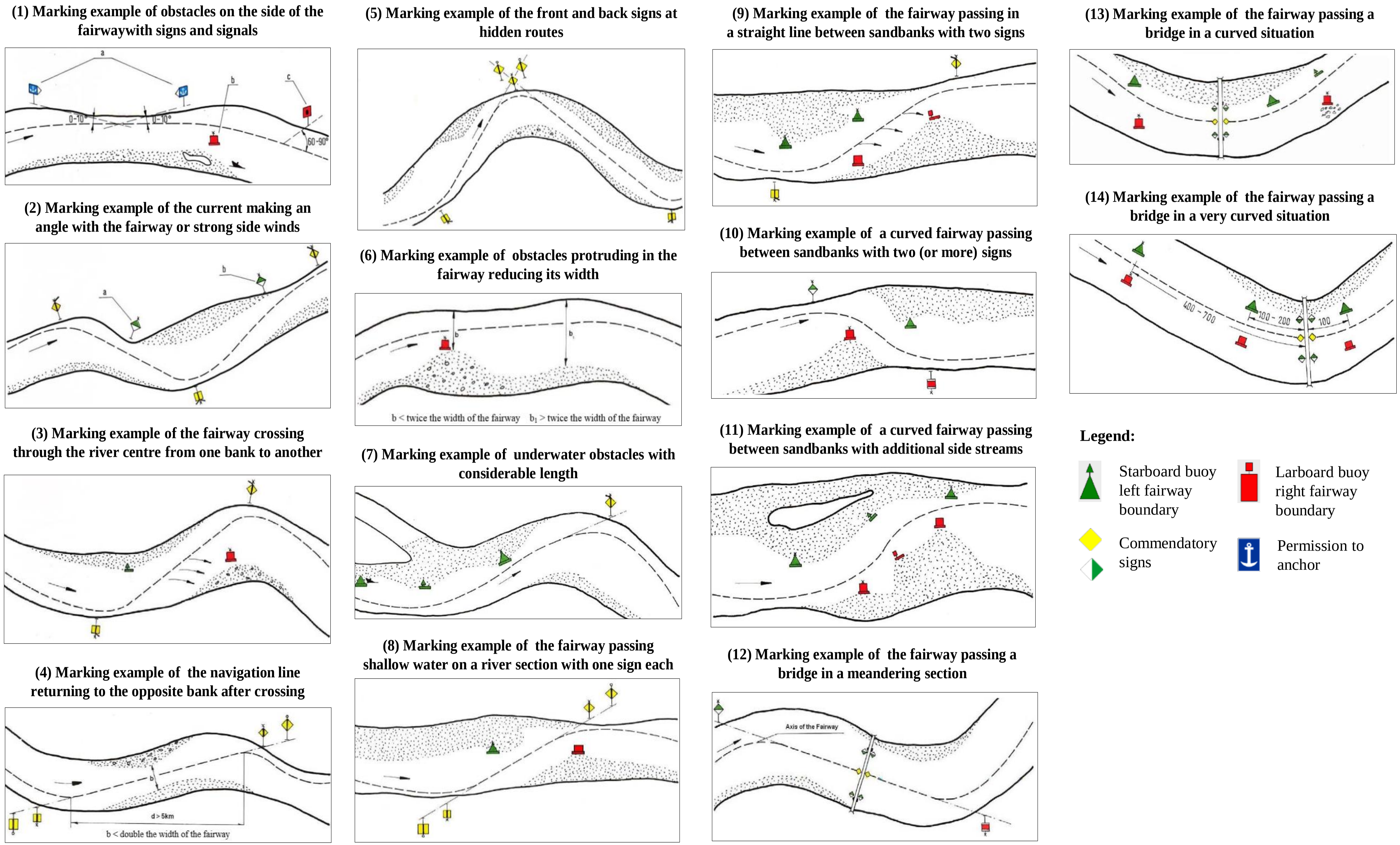


Figure 110: Marking examples of the fairway for typical situations on inland waterways according to the UNECE "Guidelines for Waterway Signs and Marking" [UNECE 2013]

5.2.3 Cost calculation for operational measures

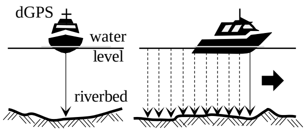
The costs of marking activities primarily consist of time-dependent personnel costs (e.g. vessel crew) and distance-dependent operation cost of marking vessels (e.g. fuel) as well as amortization costs of marking equipment. The costs of operational measures are therefore mostly determined by the length of the marking section as well as the monitoring interval with control and relocation of the position of buoys.

Because fairway marking and monitoring is a continuously ongoing process, a high amount of human resources is required. In addition to labor costs, the costs of acquisition and operation of appropriate vessels, including equipment for surveying and buoy relocation, as well as the costs of a related ICT-based software support, are significant for a modern waterway management. The costs of buoys themselves, however, are manageable, since they offer a long service life and low unit costs. The presented cost approach focuses on the main cost component of fairway operation, i.e. the survey of the riverbed in appropriate intervals including crew and equipment. The cost estimations are thereby based on data that have been collected as a part of the feasibility study for a Danube-wide implementation of an ICT-based waterway asset management system. In order to compare operational measures with the implementation of dredging measures and river engineering structures based on a life cycle approach, annual costs per survey kilometer and survey frequency must be known. The cost calculations for both single-beam and multi-beam surveys show the dominance of labor costs, especially in Austria. Furthermore, the majority of the total costs are fixed if crew and vessels are an own-and-operate approach, leading to certain budget needs, even if no kilometer of survey is performed at all. On the other hand, annual costs may be reduced significantly if free survey capacities are offered to port operators or other third parties, or if the entire survey is tendered on the market. However, the investment costs are rather high if no sufficient surveying capacity is owned by waterway operators. As a basis for the assessment of necessary equipment and staff in each riparian country, the performance and costs of cross-sectional surveys, with single-beam as well as multi-beam equipment, have to be estimated (Figure 111) [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Based on an assumed surveying speed of 4-5 km/h per hour, 150 m length of measurement of profiles and 50 m distance between profiles, it should be possible to cover 4.0 to 5.0 km survey and per day and vessel, including the necessary time from the starting point to the surveying area and back. With around 200 days per year having favorable surveying conditions and operating vessels the survey performance per vessel with single-beam can be estimated at 800 to 1,000 km per year. The calculation of investment and running costs is based on a deterministic life-cycle cost approach with a service life of 40 years for the vessel, 20 years for single-beam equipment and 10 years for computers and monitors. The calculation also includes running costs from maintenance and repair as well as insurance, taxes and fuel, based on an interest rate of 3%, leading to annual costs between € 32.000 and 34.000 per vessel per year or € 32 to 34 per kilometer of survey. The costs for staff are calculated separately due to large differences in labor costs and taxes in riparian countries based on 1 x captain and 2 x crew, with annual costs in Austria of around € 100,000 (100%). According to labor cost data from EUROSTAT, the total costs for the crew can be estimated as a fraction of these values for the riparian countries as well (e.g. SK and HU = 50%; HR, RS and RO = 40%; BG and UA =30%).The calculation of performance and costs of multi-beam surveys is conducted on a rather similar way, based on a slightly higher vessel speed of 5-7 km/h parallel to the fairway, due to a higher density and accuracy of the equipment. With an average depth of 5.0 m for regular surveys and 3.0 m for shallow sections the survey width can be estimated with 21 to 23 m (regular) and 10 to 12 m (shallow) [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

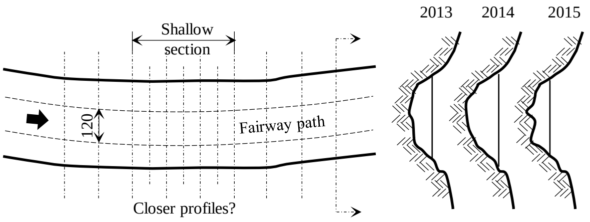
Survey of riverbed with single-beam based on cross – sectional profiles with capacity and performance

Principle: Single-beam

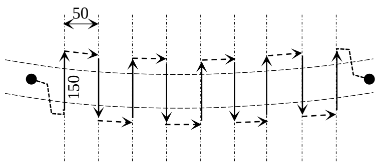


- Measurement of distance to ground
- Depth information in motion direction
- Coverage = single line

SB Survey approach: Cross-sectional profiles



SB Survey performance: [km]



Crew: 3 people, 8h/d
Approach: 25-30 km/h
Survey speed: 7-10 km/h
~ 2.5 m/s
Profiles: 150 m length every 50 m
Performance: (150 +50)/2.5 = = 80 sec + turning, adjustment → 2 min/50 m → 40 min/km

Example 1 – Performance regular survey entire river:

Approach/return to central base Ø 1.5h → Radius = 2x2x1.5 = 6h → 150 - 180 km
Breaks: 1x0.5 h/day → Ø Survey time/day = 8-2x1.5-0.5 = 4.5 h = 270 min →
Performance/day: (270 min) / (40 min/km) = 6.75 → **6.5 – 7.0 km/day**
Duration full survey 150 to 180 km from central base: 22 - 26 work days

Example 2 – Investment & running costs survey (3% interest):

Survey vessel (medium size, bare bone):	e.g. Ø 500,000 €; 40 years service life (1x)
Single beam equipment (medium):	**e.g. Ø 20,000 €; 20 years service life (2x)**
Computer, Monitors etc.:	e.g. Ø 10,000 €; 10 years service life (4x)
Maintenance & Repair.:	1.0% of investment costs per year
Vessel availability / max. performance:	200 workdays → 1,300 – 1,400 km/year
Fuel: → Ø 15-20 km/h → 0.5 l/km	1,400 km x 0.5 l/km x 1.40 €/l ~ 980 €/year
Insurance, tax, miscellaneous	0.5% of investment costs per year
Crew (e.g. AT): 2x30,000; 1x40,000 captain (DE/AT=100%; SV/HU=50%; HR/RS/RO=40%; BG/UA=30%)	100,000 €/a labour costs for crew (100%)

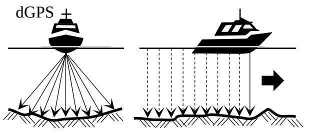
Option single-beam - regular (50 m cross-sectional profiles):

Interest rate [%] 3.0% Duration cycle 40

Item	*Costs*	*Time*	*Present value*	*Annual costs*
Vessel	500,000.0	0	500,000.0	21,631.2
Single-beam 1	20,000.0	0	20,000.0	865.2
Single-beam 2	20,000.0	20	11,073.5	479.1
Computer, Monitor 1	10,000.0	0	10,000.0	432.6
Computer, Monitor 2	10,000.0	10	7,440.9	321.9
Computer, Monitor 3	10,000.0	20	5,536.8	239.5
Computer, Monitor 4	10,000.0	30	4,119.9	178.2
Maintenance (1%)	5,300.0	per year	122,508.3	5,300.0
Insurance, tax (1%)	2,650.0	per year	61,254.1	2,650.0
Fuel	980.0	per year	22,652.5	980.0
Total without crew			**764,586.0**	**33,077.8**
Survey vessel costs per survey [€]/km				**23.6**
Crew max (100%)	**100,000.0**	**per year**	**2,311,477.2**	**100,000.0**
Crew costs per survey [€/km]				**71.4**
Crew min (30%)	**30,000.0**	**per year**	**693,443.2**	**30,000.0**
Crew costs per survey [€/km]				**21.4**

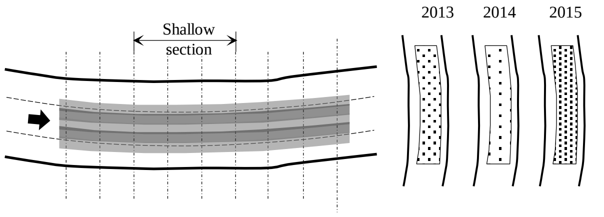
Survey of riverbed with multi-beam based overlapping stripes with capacity and performance

Principle: Multi-beam

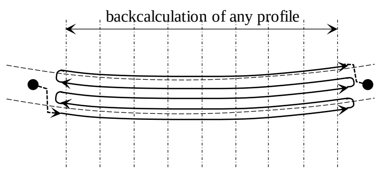


- Measurement of distance to ground
- Depth information in motion direction
- Coverage = stripe

MB Survey approach: Overlapping parallel stripes



MB Survey performance: [km]



Crew: 3 people, 8h/d
Approach: 25-30 km/h
Survey speed: 7-10 km/h
~ 2.5 m/s
Width shallow: Ø depth = 3.0 m
draught + squat =0.5 m
135° Angle → ~ 11 m
Width total: Ø depth = 5 m
draught + squat =0.5 m
135° Angle → ~ 22 m

Example 1 – Performance regular survey entire river:

Approach/return to central base Ø 1.5h → Radius = 2x2x1.5 = 6h → 150 - 180 km
Breaks: 1x0.5 h/day → Ø Survey time/day = 8-2x1.5-0.5 = 4.5 h = 270 min →
Performance/day: 150/(22) ~ 7x → (2.5/7)*60*270/1,000 = **5.5 – 6.0 km/day**
Duration full survey 150 to 180 km from central base: 26 - 31 work days

Example 2 – Investment & running costs survey (3% interest):

Survey vessel (medium size, bare bone):	e.g. Ø 500,000 €; 40 years service life (1x)
Multi beam equipment (medium):	**e.g. Ø 250,000 €; 20 years service life (2x)**
Computer, Monitors etc.:	e.g. Ø 15,000 €; 10 years service life (4x)
Maintenance & Repair.:	1.0% of investment costs per year
Vessel availability / max. performance:	200 workdays → 1,100 – 1,200 km/year
Fuel: → Ø 15-20 km/h → 0.5 l/km	1,200 km x 0.5 l/km x 1.40 €/l ~ 840 €/year
Insurance, tax, miscellaneous	0.5% of investment costs per year
Crew (e.g. AT): 2x30,000; 1x40,000 captain (DE/AT=100%; SV/HU=50%; HR/RS/RO=40%; BG/UA=30%)	100,000 €/a labour costs for crew (100%)

Option multi-beam - regular (width = 20-22 m coverage):

Interest rate [%] 3.0% Duration cycle 40

Item	*Costs*	*Time*	*Present value*	*Annual costs*
Vessel	500,000.0	0	500,000.0	21,631.2
Multi-beam 1	250,000.0	0	250,000.0	10,815.6
Multi-beam 2	250,000.0	20	138,418.9	5,988.3
Computer, Monitor 1	15,000.0	0	15,000.0	648.9
Computer, Monitor 2	15,000.0	10	11,161.4	482.9
Computer, Monitor 3	15,000.0	20	8,305.1	359.3
Computer, Monitor 4	15,000.0	30	6,179.8	267.4
Maintenance (1%)	7,650.0	per year	176,828.0	7,650.0
Insurance, tax (1%)	3,825.0	per year	88,414.0	3,825.0
Fuel	840.0	per year	19,416.4	840.0
Total without crew			**1,213,723.7**	**52,508.6**
Survey vessel costs per survey [€]/km				**43.8**
Crew max (100%)	**100,000.0**	**per year**	**2,311,477.2**	**100,000.0**
Crew costs per survey [€/km]				**83.3**
Crew min (30%)	**30,000.0**	**per year**	**693,443.2**	**30,000.0**
Crew costs per survey [€/km]				**25.0**

Figure 111: Performance and cost estimation for single-beam surveying based on cross-sectional profiles compared to multi-beam surveying based on overlapping parallel swathes [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Thus, water depth is crucial for getting a full picture based on overlapping longitudinal surveying swathes and the resulting daily performance of around 3.7 to 4.0 km (regular) respectively 1.9 to 2.0 km (shallow).

As a result, the regular survey performance with multi-beam equipment (700 to 800 km per year) is lower compared to single-beam, but offers a much higher accuracy and information density. With labor costs staying the same, the differences in surveying costs per kilometer are mainly related to higher costs of the equipment (€ 50,000 to 55,000 per year) and lower surveying performance. As a result, the annual cost for riverbed surveying with single-beam equipment (assuming a distance of 50 meter between cross-sectional profiles) amount to 23.6 €/rkm without staff. If the labor costs of crew members are included as well, annual costs of single-beam riverbed surveying range between 95.0 €/rkm (DE/AT) and 45.0 €/rkm (BG/UA), depending on the respective labor costs in individual riparian countries. For multi-beam riverbed surveying (assuming a coverage of 20-22 m of width), the annual costs without staff are with 43.8 €/rkm, which is noticeably higher. When labor costs are included as well, arising annual costs for multi-beam surveying are between 127.1 €/rkm (DE/AT) and 68.8€/rkm (BG/UA), as illustrated in Figure 111. Figure 112 and Figure 113 provide information on cost estimations for fairway operation for all riparian Danube countries comparing single- and multi-beam riverbed surveys also accounting for the length of national river stretches and critical sections. Finally, the total annual costs for both crew and vessel, based on minimum and recommended equipment/survey capacity, are given as well. The resulting total costs of € 1.6 million per year for single-beam surveys respectively € 2.0 million per year for multi-beam surveys, of the entire Danube (covering around 2,400 km) are rather low. With the difference in costs between multi-beam and single-beam, additional possibilities and accuracy of multi-beam surveys, at least one of the latter, should be available to each waterway agency. However, having both devices on each vessel is considered as favorable. Despite low annual costs for each waterway agency for such equipment, it has to be noted that investment costs in the year of acquisition may be a rather prohibitive factor with current tight budgets of most agencies. In addition, current available equipment in most agencies is rather old, leading to certain investment needs that are a necessary prerequisite for providing accurate and actual information on fairway conditions in the future [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Cost calculation and needs for regular riverbed surveying in riparian Danube countries: Scenario 1: 1x year full, +5x critical with Single-beam

Country*	*lenght_l [km]*	*lenght_r [km]*	*total [km]*	*critical km]***	*survey_reg [1/year]*	*survey_crit [1/year]*	*tot_survey [km/a]*	*Vessels_min [-]*	*Vessels_rec. [-]****	*crew_cost factor [%]*	*costs_min [-]*	*costs_rec. [-]****
Germany	213.0	191.0	202.0	72.8	1	5	930.0	1	2	100%	132,798	265,596
Austria	322.0	350.0	336.0	14.7	1	5	482.5	1	2	100%	132,798	265,596
Slovakia	172.0	62.0	117.0	7.7	1	5	193.5	1	1	50%	82,798	82,798
Hungary	275.0	378.0	326.5	31.9	1	5	645.5	1	2	50%	82,798	165,596
Croatia	0.0	138.0	69.0	42.3	1	5	492.0	1	2	40%	72,798	145,596
Serbia	358.0	450.0	404.0	80.3	1	5	1207.0	2	3	40%	145,596	218,393
Romania	941.0	374.0	657.5	48.16	1	5	1139.1	2	3	40%	145,596	218,393
Bulgaria	0.0	472.0	236.0	8.75	1	5	323.5	1	2	30%	62,798	125,596
Ukraine	133.0	0.0	66.5	22.2	1	5	288.5	1	2	30%	62,798	125,596
										Total	920,776	1,613,158

*Moldova with a lenght of 0,55 km (l) is covered by Romania ***Recommendation based on fast parallel survey / redundancy needs
** Joint border sections, critical sections partly joined/alternating survey (counted 50:50)

Figure 112: Minimum and recommended equipment for a sufficient riverbed survey with single-beam together with an estimation of related annual costs in each riparian country [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Cost calculation and needs for regular riverbed surveying in riparian Danube countries: Scenario 1: 1x year full, +5x critical with Multi-beam

Country*	*lenght_l [km]*	*lenght_r [km]*	*total [km]*	*critical km]***	*survey_reg [1/year]*	*survey_crit [1/year]*	*tot_survey [km/a]*	*Vessels_min [-]*	*Vessels_rec. [-]****	*crew_cost factor [%]*	*costs_min [€/year]*	*costs_rec. [€/year]****
Germany	213.0	191.0	202.0	72.8	1	5	930.0	2	2	100%	304,457	304,457
Austria	322.0	350.0	336.0	14.7	1	5	482.5	1	2	100%	152,229	304,457
Slovakia	172.0	62.0	117.0	7.7	1	5	193.5	1	1	50%	102,229	102,229
Hungary	275.0	378.0	326.5	31.9	1	5	645.5	1	2	50%	102,229	204,457
Croatia	0.0	138.0	69.0	42.3	1	5	492.0	1	2	40%	92,229	184,457
Serbia	358.0	450.0	404.0	80.3	1	5	1207.0	2	3	40%	184,457	276,686
Romania	941.0	374.0	657.5	48.2	1	5	1139.1	2	3	40%	184,457	276,686
Bulgaria	0.0	472.0	236.0	8.8	1	5	323.5	1	2	30%	82,229	164,457
Ukraine	133.0	0.0	66.5	22.2	1	5	288.5	1	2	30%	82,229	164,457
										Total	1,286,743	1,982,343

*Moldova with a lenght of 0.55 km (l) is covered by Romania ***Recommendation based on fast parallel survey / redundancy needs
** Joint border sections, critical sections partly joined/alternating survey (counted 50:50)

Figure 113: Minimum and recommended equipment for a sufficient riverbed survey with multi-beam together with an estimation of related annual cost in each riparian country [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

5.3 River engineering measures

For natural water bodies, a variety of river engineering measures aiming at different objectives can be applied. Some of them have purely ecological backgrounds, while others intend to improve flow conditions, stabilize the riverbed or serve for river bank protection, and still others are implemented to improve conditions for inland navigation. River engineering measures aiming at improved fairway conditions include different types of groins and staging, among others. Since groins represent the most common river engineering measure executed on the Austrian Danube stretch, they will be examined more in detail in terms of waterway asset management on behalf of all engineering measures, also including basic approaches for economic assessment. A combined implementation of groin fields and stabilization of the riverbed leads to a sectional narrowing of the flow area, subsequently resulting in a permanent increase of the water level (Figure 114).

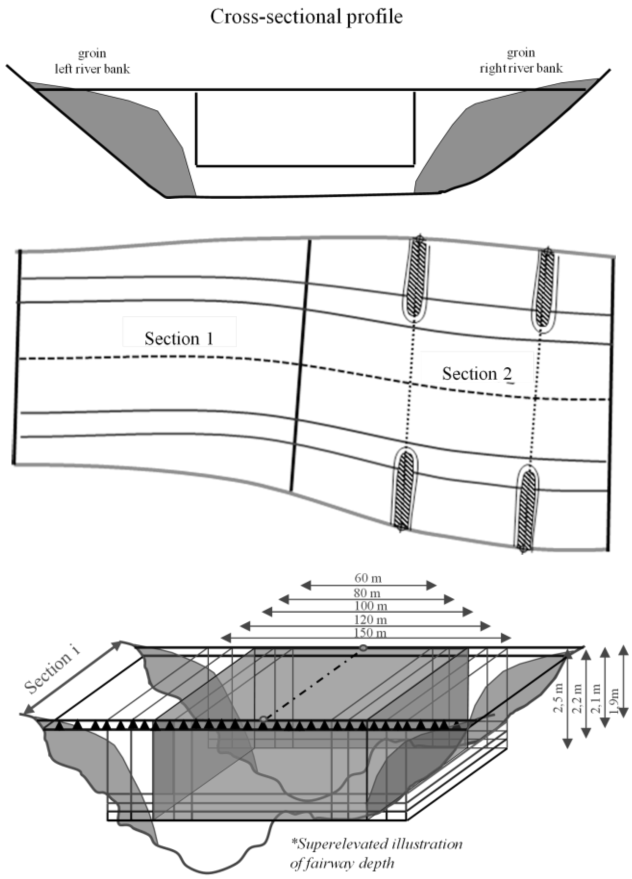


Figure 114: Visualization of groins in cross section and top view and principle of average elevation of water level through the construction of groins

As mentioned afore, a variety of groins exist, which significantly differ regarding their impact on flow conditions, sedimentation and erosion processes. Due to the construction of groins, increased erosion of the riverbed in central fairway areas occurs, requiring a stabilization of the riverbed in some cases. Important parameters for classification of groin fields are the ratio of river width and length of groins, their angle of inclination, the distance between individual groins, the total length of a groin field and the related water level for dimensioning of groin elevation. Relevant for the description of flow conditions and rising of the water level is the distinction between overflowed and not overflowed groins. For the evaluation of resulting water level elevation, different lengths of groin fields (overflowed and non-overflowed) were compared to each other as well as different groin lengths and different distances between groins within a study presented by the Institute of Hydraulic Engineering. For the one dimensional calculation of resulting water level elevation, the software package HEC-RAS was used. Another modelling was performed with HydroAS-2D software for two-dimensional flow modeling. The resulting water level elevation for overflowed and not overflowed groins were compared to each other. Results indicated the two-dimensional calculations provide more stable results, especially for overflowed groins [Krouzecky, N. et al 2015]. Assessments of the impact of groins should therefore, in any case, be based on 2D-flow calculations. Generally, river engineering measures cause higher construction costs compared to maintenance measures, but have a higher duration of impact as well. For the economic assessment of groins, annual costs of groins can be calculated using initial constructions costs as well as an estimated service life of 30 and 50 years. Thereby, the construction costs of groins can be determined using market prices for block stones and a volume calculation based on their cross-sectional dimensions, lengths and the number of groins. In practice, factors, such as planning and construction time as well as environmental restrictions, significantly affect resulting measure costs and must therefore be evaluated in a more detailed process.

5.4 Maintenance measures

With the implementation of maintenance measures, i.e. dredging a "deep fairway channel" for the full width of the fairway, the geometry of the riverbed is modified. This modification is characterized by a (sudden) difference in riverbed altitudes on the time scale and may be verified by single- or multi-beam riverbed surveys (compare Figure 117). Depending on the existing regulations, the maximum dredging depth may be limited as well, whereas necessary minimum dredging depth is determined on the basis of targeted availability levels.

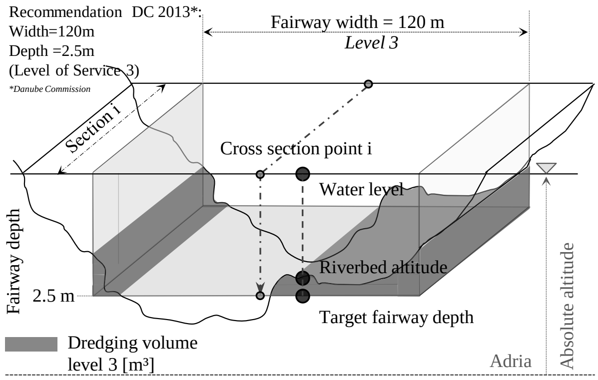


Figure 115: Cross section point i

The riverbed geometry available after dredging works should lead to improved fairway availability throughout the year, according to Figure 115 and Figure 116 (dark grey). Naturally, sedimentation and erosion are continuous processes, with the duration of measure impact being defined by the time until the dredged volume will have filled back and predicted fairway availability dropped to the level of initial availability performance without measures. If other parameters of this process are recorded in the database as well, such as corresponding discharge level, flow speed or average grain size, then further statistical analyses and even more accurate empirical predictions might become feasible. Typically, an assessment of implemented measures has to cover the entire timeframe prior to implementation until the end of the impact time. While cost information of measures, might be available shortly after implementation, the time of impact may be very long. For a comparison of different measures, both costs and time of impact need to be known. As a first approach, especially for measures with a very long time of impact, expert guesses could be a starting point until the necessary information can be obtained. Another approach would be a backward assessment of implemented measures during the last years or decades, provided that the necessary information regarding development of the riverbed geometry (single- or multi-beam surveys) is still available.

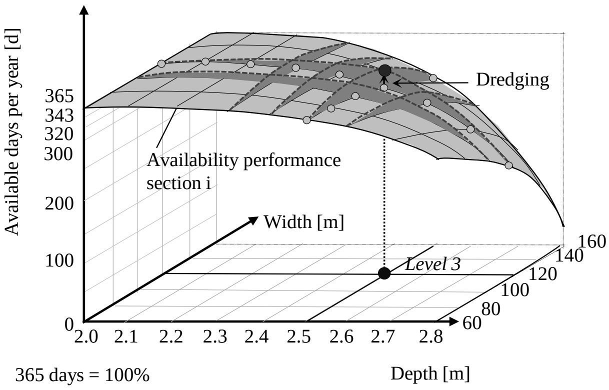


Figure 116: Fairway availability before and after dredging

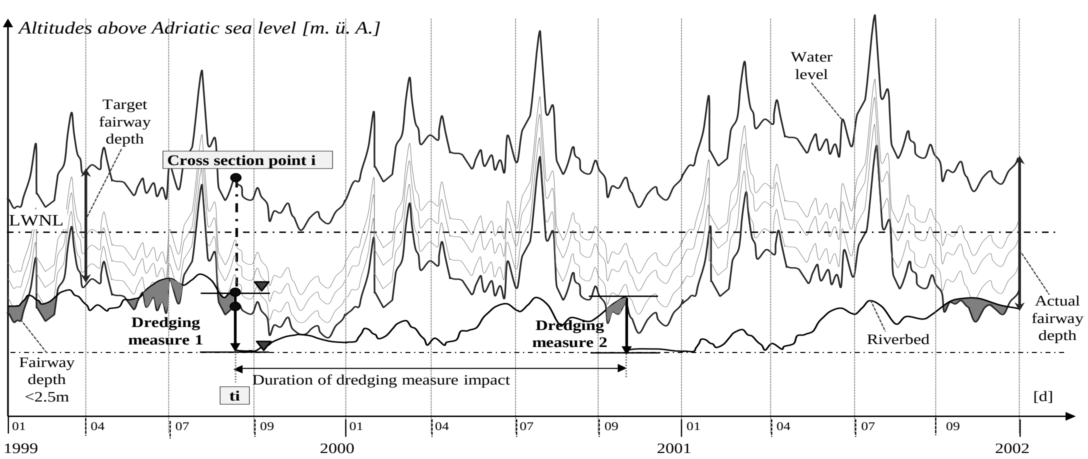


Figure 117: Development of absolute riverbed altitude, water level and resulting fairway depth at cross section point including the impact of dredging measures [Haselbauer, K. et al. 2014]

5.5 Overview of economic assessment of dredging measures

Every planned measure leads to costs that need to be covered either within budget range of the responsible waterway authority or within another funding scheme. Depending on the particular geometry of a river section, a specific dredging volume is necessary in order to achieve any given target fairway width and depth. With increasing target fairway parameters, the required dredging volume is increasing as well and leads to increasing total costs of a dredging measure (compare Figure 131 and Figure 129).

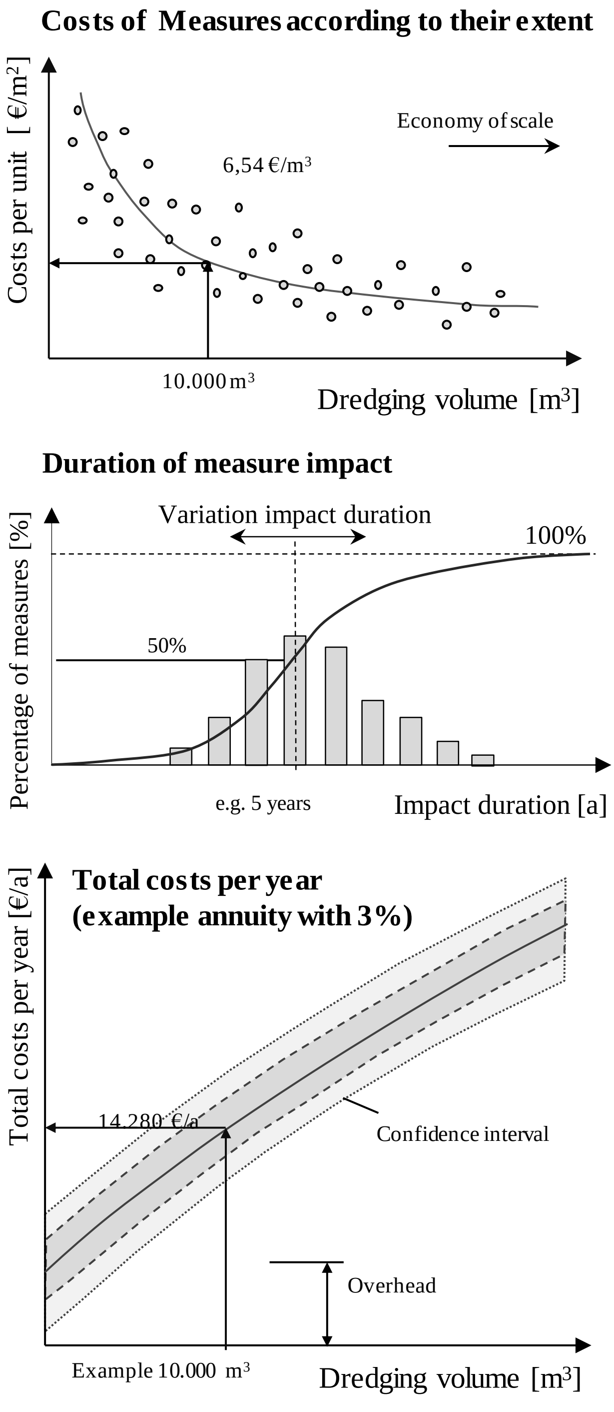


Figure 118: Calculation of total annual costs for dredging based on actual measure extent, unit costs and duration of measure impact.

The economics of scale apply for all measures in waterway asset management. Figure 118 provides an exemplary overview of decreasing dredging costs (fine sediment) with increasing measure extent for the Austrian section of the Danube. Thus, in the time period from 2009 to 2013, the average dredging costs per unit for fine sediment and gravel amounted to € 6.54 per m³ and € 8.36 per m³ respectively (also compare Figure 130 and Figure 133).

In a life cycle approach, a comparison of measure efficiency has to include both costs and duration of measure. The duration of dredging measure impact shows a certain variation, as illustrated in Figure 118, and is further described in section 5.5.5. If all implemented dredging measures, including related riverbed surveys before and after measure as well as the respective dredging volumes, are systematically incorporated in an asset management data base, a backwards-oriented analysis of impact duration including backfilling curves of dredging volume (as presented in Figure 81) and prevailing discharge amounts becomes feasible for each shallow section. A first analysis of the statistics of dredging measures for the Austrian Danube stretch between 2009 and 2012 indicates an average intervention rate of 0.43 interventions per ford per year. Thus, on average, dredging measures at shallow sections provide a duration of measure impact of around 2.5 years (compare Figure 136). For an economic comparison of measures, annual costs of a measure (annuity) are calculated based on impact duration and measure costs using the equations provided in chapter 2.3.14. Figure 118 provides an example of the development of annual costs of dredging measures, depending on the respective dredging volume. The following sections will provide an overview of both, the practical planning and implementation procedure, as well as the methodological approach.

5.5.1 Practical approach to planning of dredging measures

In the case of a limited budget and/or limited available dredging capacity on the local market, necessary measures may not be implemented within the available budget or dredging period. In order to continuously maintain availability at a high level for an entire river stretch, a priority ranking for the implementation of measures is required. Prior to a possible occurrence of low water levels, e.g. during winter months, all potential critical sections are surveyed using multi-beam surveying equipment. The following evaluation of available fairway depths at critical sections is based on the last riverbed survey. In a next step, priority is given to those critical sections showing the lowest available fairway depths. The prioritization process can be carried out for different levels of service, allowing flexible adjustments of fairway maintenance works in terms of traffic volume and available measure budget.

Figure 119: LOS-related planning of dredging measures at ford Weissenkirchen based on riverbed surveys before and after measures

In order to implement a dredging measure for one critical section, the calculation of dredging volume is a prerequisite. With the aim to minimize deviations between calculated dredging needs and actual dredging volume, results of riverbed surveys not older than one month should be used. Depending on the targeted Level of Service (LOS 1, LOS 2, LOS 3), the necessary width and depth of the dredging area can be determined and may be automatically displayed in a WAMS as a suggested dredging polygon. In a further optional step, the manual optimization of the dredging measure, based on changes in the shape of the dredging polygon and target depth, allows for accounting for individual local circumstances. The dredging module of a WAMS should be capable of displaying all necessary results of planned dredging measures. The necessary dredging volume for various target levels is the result of a comparison of the actual riverbed geometry, the dredging area and the target dredging depth. As this is a standard task, the developed WAMS software of VIADONAU is capable of performing these calculations as well as others with ease. For a precise determination of required dredging volume, triangulated irregular networks (TIN) data of the riverbed surface from processed multi-beam surveying data should be used [Hoffmann, M. & Haselbauer, K.

& Blab, R. & Hartl, T. 2014]. With unit cost functions and dredging volume at hand, an estimation of dredging costs can be displayed in real-time for any target fairway parameters. Figure 120 provides a principal overview of required dredging volume, total dredging costs, necessary time for measure implementation and estimated duration of measure impact for increasing fairway widths (Levels of Service). While measure costs and dredging volume are more or less fixed average values (with a rather small deviation), duration of measure impact is related to the specific hydraulic characteristics of a specific section. As a first approach, predictions of measure impact duration may be derived from an averaged master function based on an evaluation of already implemented maintenance measures. Further improvements of these predictions may either be based on a more thorough statistical analysis or on calculations of sedimentation processes with the use of an appropriate software solution (e.g. Flow 3D) [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

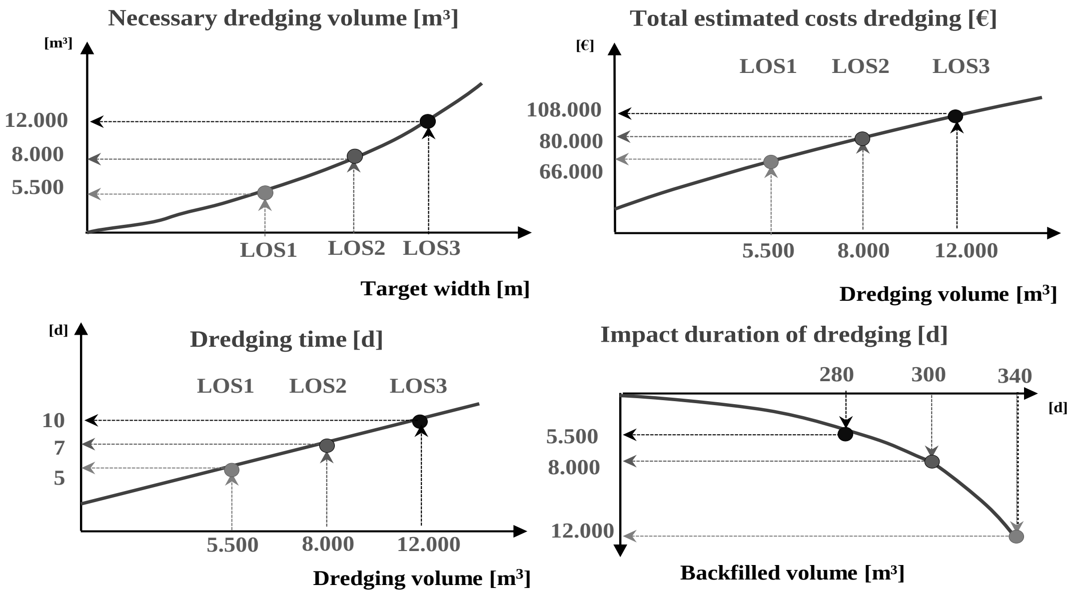


Figure 120: Dredging volume, dredging costs, dredging time and duration of measure impact for different levels of service.

The dredging time, in general, depends on the amount of dredging volume and performance of the dredging equipment, with total dredging time being a result of dredging volume divided by dredging performance in days. In addition, there are some restrictions as to the use of certain dredging equipment, such as maximum draught and flow velocity, which have to be considered. In low-water periods, hopper barges cannot be fully loaded so that the number of required trips to transport the same amount of dredging volume increases substantially together with resulting costs. In order to ensure a continuous depth of the fairway in the case of impending low-water periods, dredging a "deep fairway channel" at the beginning may be an option to assure the continuity of a certain fairway depth. If necessary, a subsequent completion of dredging to full fairway widths may increase the transport capacity. In most cases, the duration of measure impact for specific critical sections will be based on an analysis of previous measure implementations.

For a WAMS, the statistical analysis of historical backfilling behavior is proposed as an empiric solution due to certain deviations of analytical approaches with common software from actual development. If these input factors are known for a number of critical river sections, a fast estimation of dredging costs, dredging time and duration of measures impact can be provided. Furthermore, the system allows for an assessment of already implemented measures providing a continuous update of parameters [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

5.5.2 Dredging polygon for different LOS

The following chapters will provide an overview of the already completed software- technical implementation of the methodical approach for planning and optimization of dredging measures as described in section 5.5.1. In order to create a dredging polygon for a shallow section using WAMS-software, a current survey of the riverbed, represented by TIN-data, is selected and loaded onto the software to provide an overview of fairway depths related to LNWL. Figure 121 to Figure 125 give examples of the visualization of TIN-data of the riverbed, for shallow section Petronell-Witzelsdorf, for different target fairway widths.

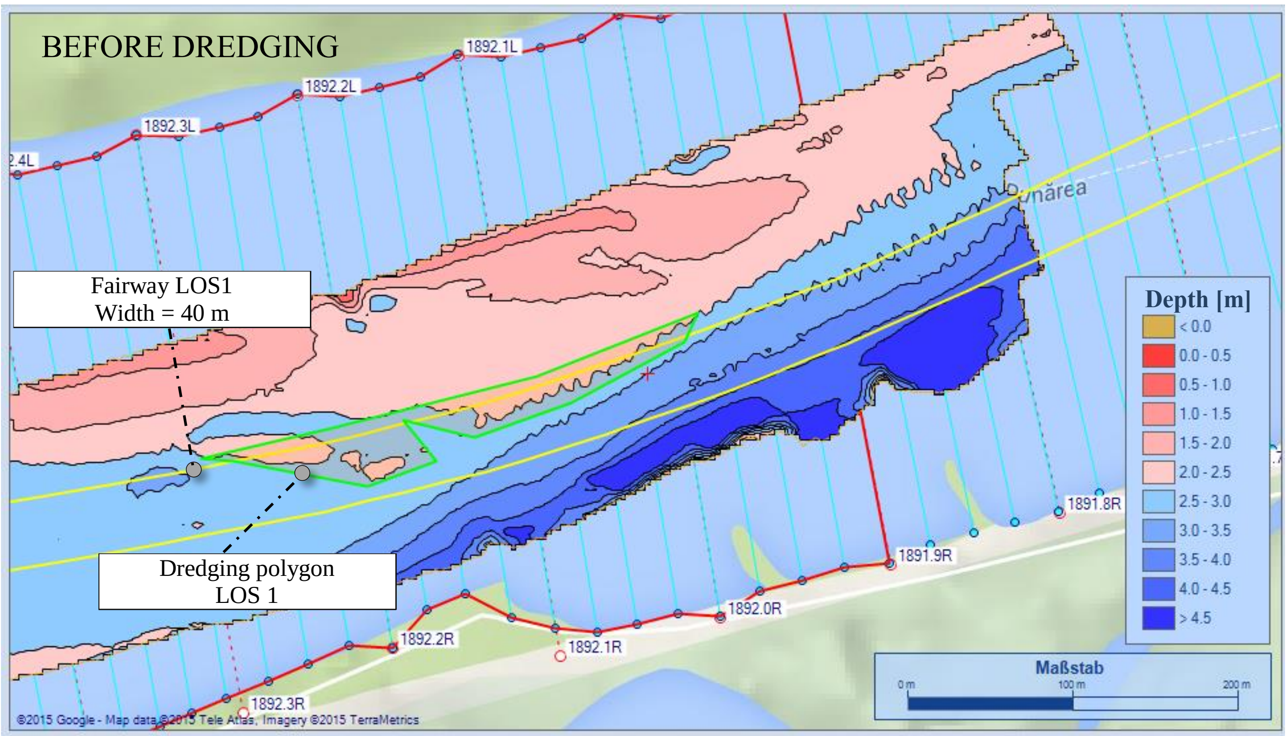


Figure 121: WAMS-example dredging measure; dredging polygon to achieve LOS1 at shallow section Petronell-Witzelsdorf based on multi-beam TIN-data of the riverbed (22.10.2014) related to LNWL2010

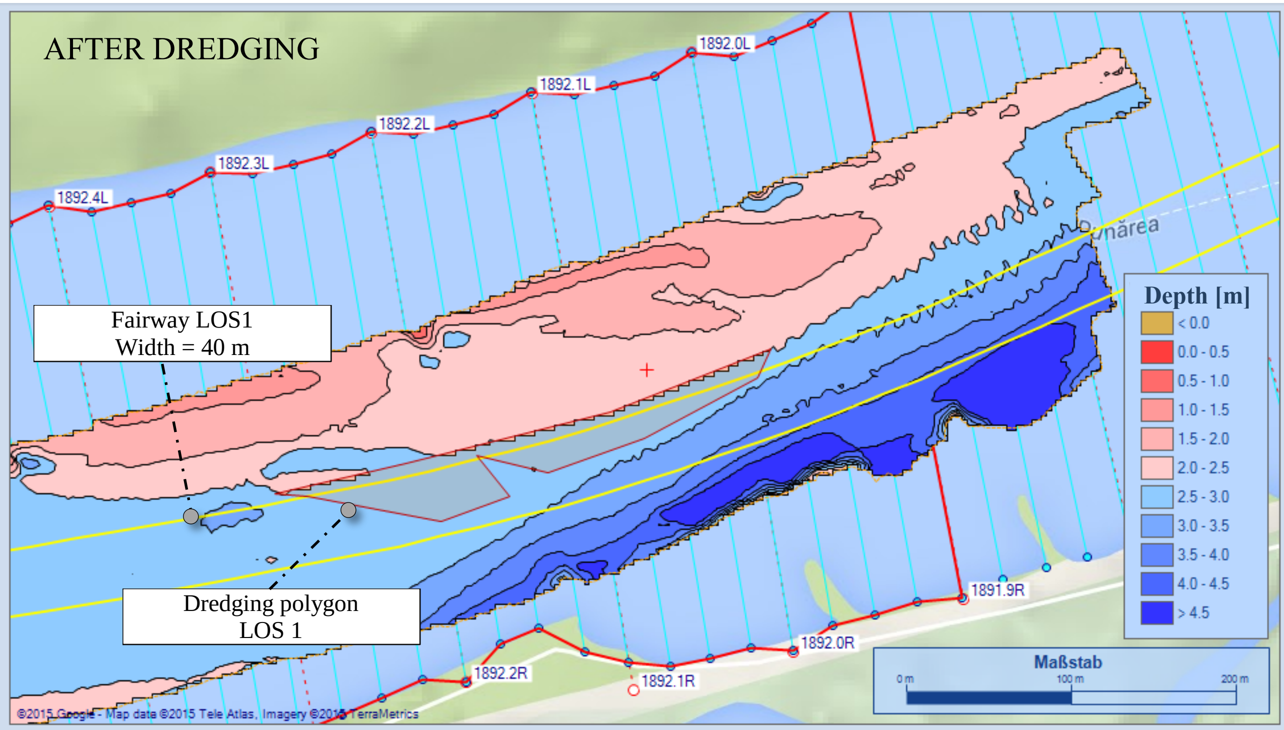


Figure 122: WAMS-example dredging measures: riverbed after dredging; dredging extent for LOS1 is derived by the dredging polygon and a target depth of 2.5 meters below LNWL based on the presented NTF-method for volume calculation using TIN-data

Based on these maps and defined fairway widths, a fast identification of areas with insufficient fairway depths is possible. As a result, a dredging polygon can be defined either manually or automatic by the software. In a manual mode, a polygon, which can consist of a flexible number of points, can be drawn around areas with inadequate depths and provide the basis for the calculation of a dredging area. The dredging area, which is bounded by the dredging polygon, can be assigned to any target depth related to LNWL. The resulting volume is subsequently intersected with the TIN-surface of the riverbed, forming the foundation for the calculation of necessary dredging volumes as described in section 5.5.3

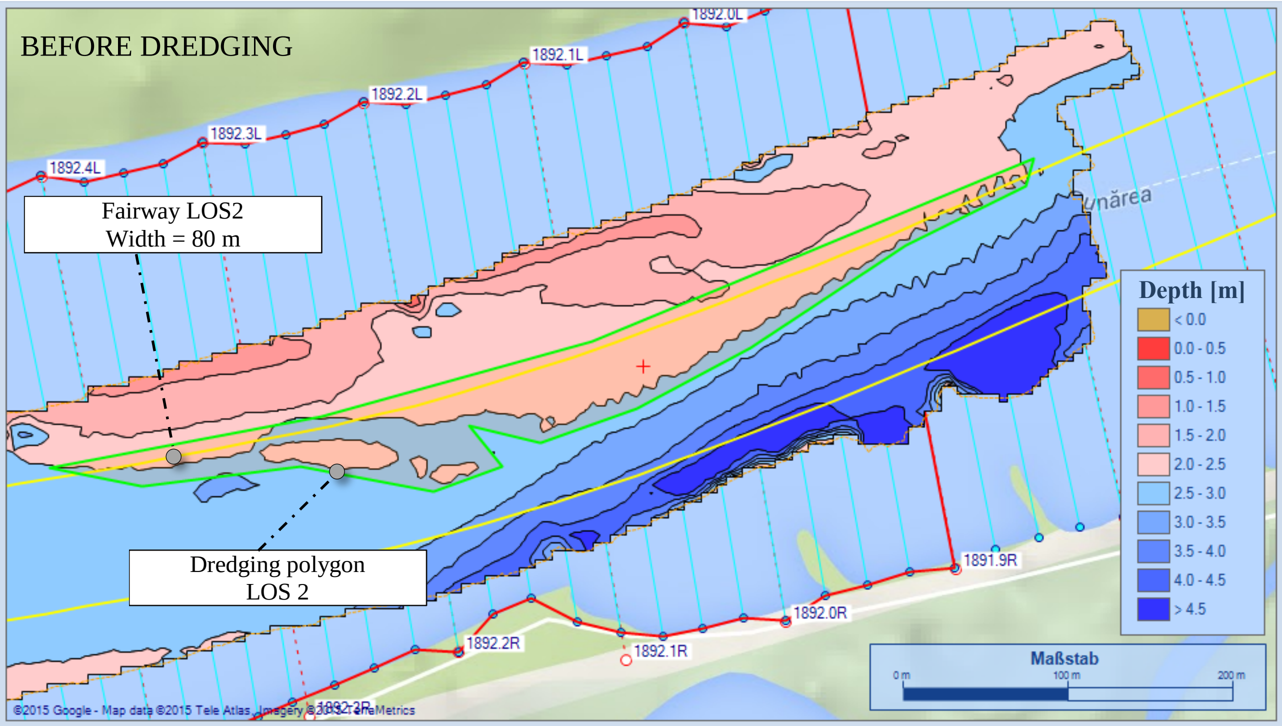


Figure 124: WAMS-example dredging measure; dredging polygon to achieve LOS2 at shallow section Petronell-Witzelsdorf based on multi-beam TIN-data of the riverbed (22.10.2014) related to LNWL2010

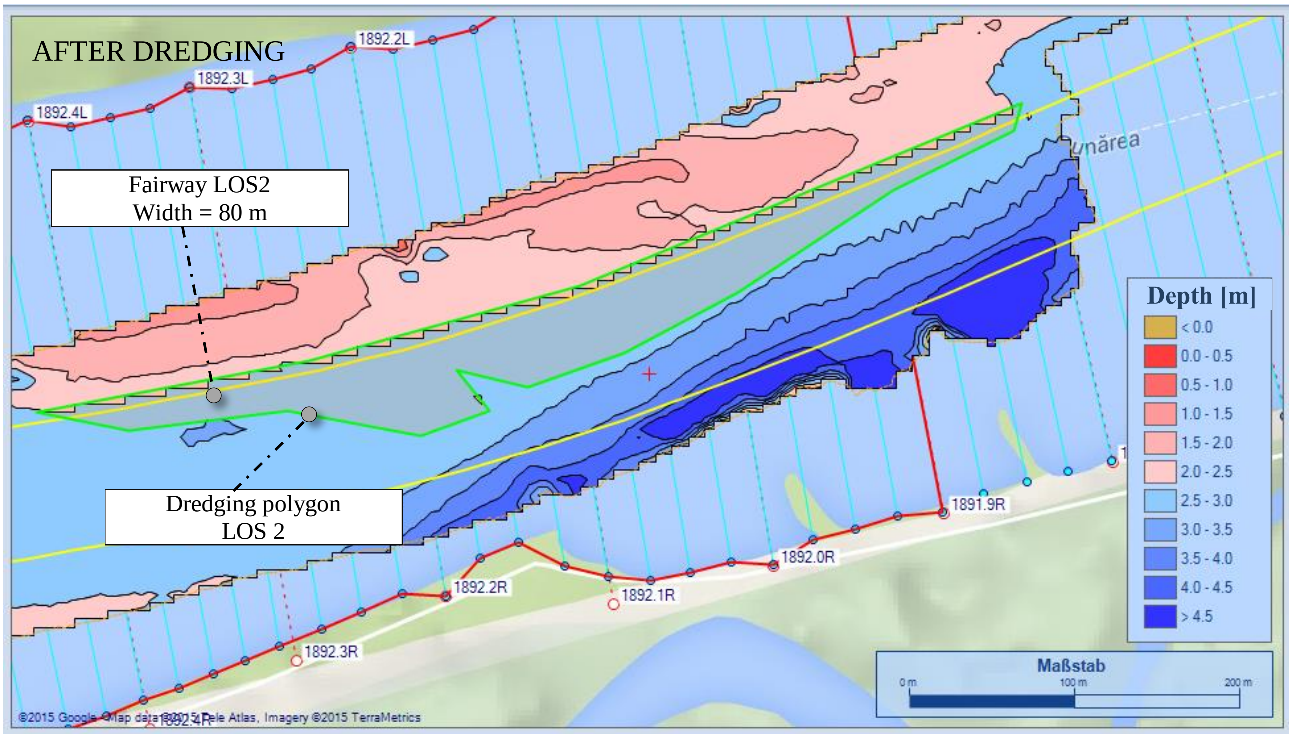


Figure 123: WAMS-example dredging measures: riverbed after dredging; dredging extent for LOS2 is derived by the dredging polygon and a target depth of 2.5 meters below LNWL based on the presented NTF-method for volume calculation (TIN)

Consequently, the total costs of each measure can be determined by applying cost functions for dredging of fine sediment or gravel (compare section 5.5.4). Figure 121 (before dredging) and Figure 122 (after dredging) indicate the fairway depths (related to LNWL) for shallow section Petronell-Witzelsdorf and a given fairway width LOS1 (40m) as well as the resulting dredging polygon. Figure 124 and Figure 125 clearly indicate that the dredging area substantially increases with increasing fairway width, resulting in correspondingly high dredging volumes. The algorithm for the calculation of measures extent will be presented in the following chapter 5.5.3.

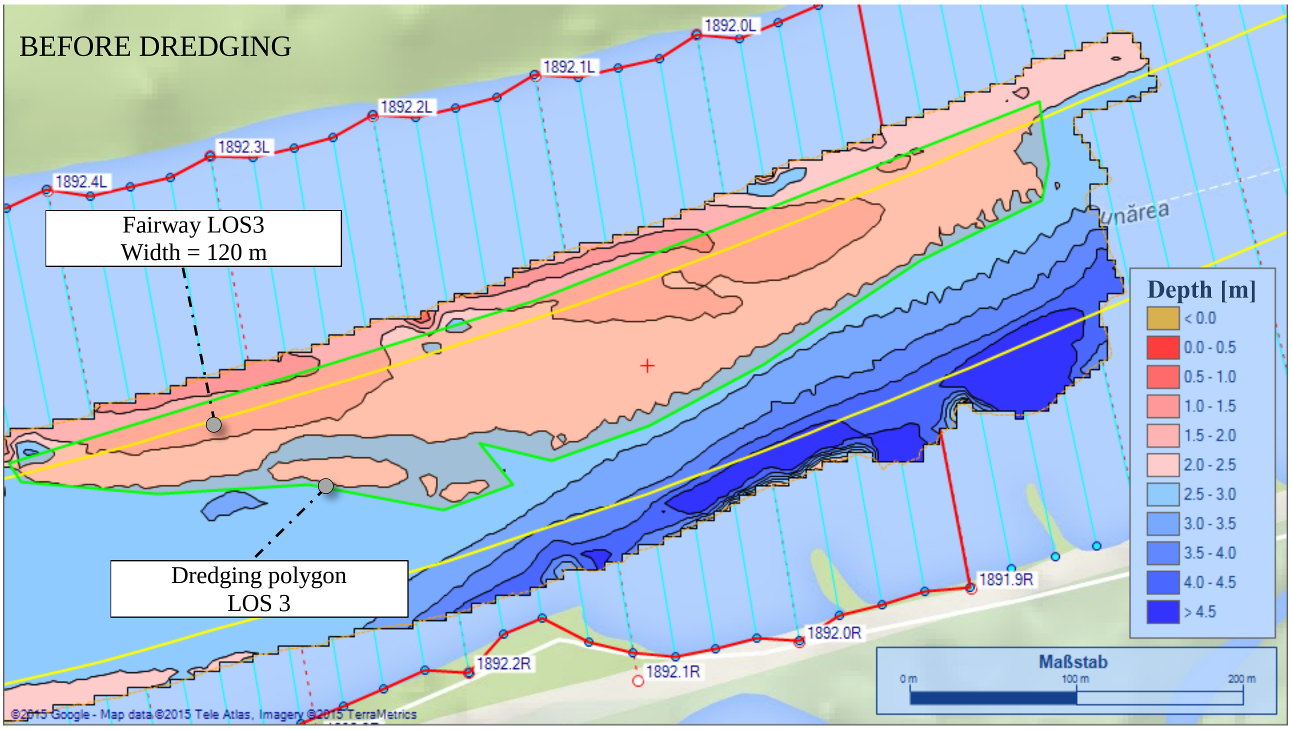


Figure 125: WAMS-example dredging measure; dredging polygon to achieve LOS3 at shallow section Petronell-Witzelsdorf based on multi-beam TIN-data of the riverbed (22.10.2014) related to LNWL2010

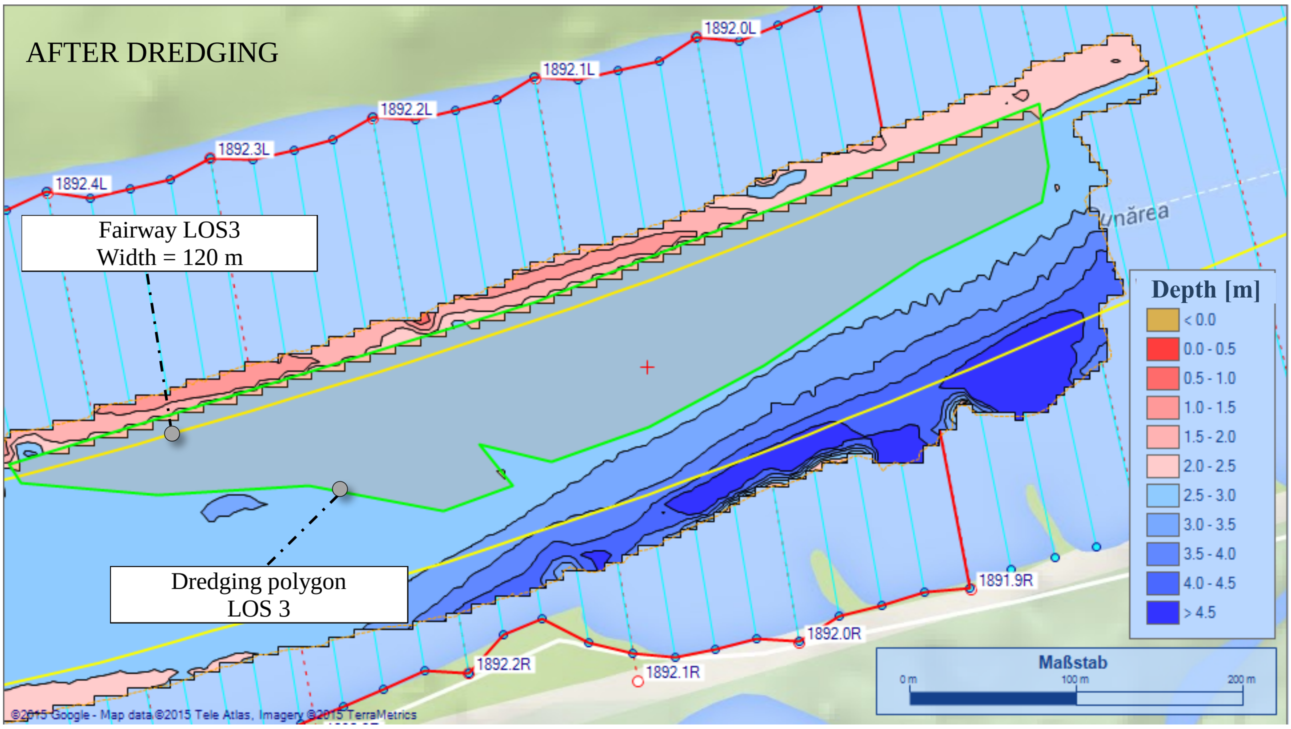


Figure 126: WAMS-example: dredging measures: riverbed after dredging; dredging extent for LOS3 is derived by the dredging polygon and a target depth of 2.5 meters below LNWL based on the presented NTF-method for volume calculation using TIN-data

5.5.3 Calculation of measure extent

Depending on the particular geometry of a river section, a specific dredging volume is necessary in order to achieve any given target fairway width (LOS1, 2, 3) and depth. With increasing target fairway parameters, the required dredging volume is increasing as well, resulting in a concave rising dredging volume surface (Figure 128).

The volume calculation in the presented approach is based on TIN-data, because tests with raster data revealed substantial deviations that were closely related to the shape of the polygon and the geometry of the shallow section. Since the resulting dredging volume is the basis for further cost estimations and is also essential for billing, a reliable and accurate algorithm had to be found.

For the calculation of dredging volumes, it is essential to define appropriate reference levels. These reference levels are provided by the definition of a target related to LNWL depths for dredging. By modeling the surface of the riverbed as a triangulated irregular network, the entire volume can be described very precisely by the sum of individual triangular prisms, as illustrated in Figure 127 [Vulic, M. et al. 2006]. With this process, the volume of a limited surface space is separated by volume calculation of the final number of vertical triangle prisms. Each triangular prism can be divided into several geometric shapes that may be easily described mathematically. The resulting shapes are two tetrahedrons and one regular prism. The volume of the regular prism V_{Prism} may be calculated with equations (52) and (53):

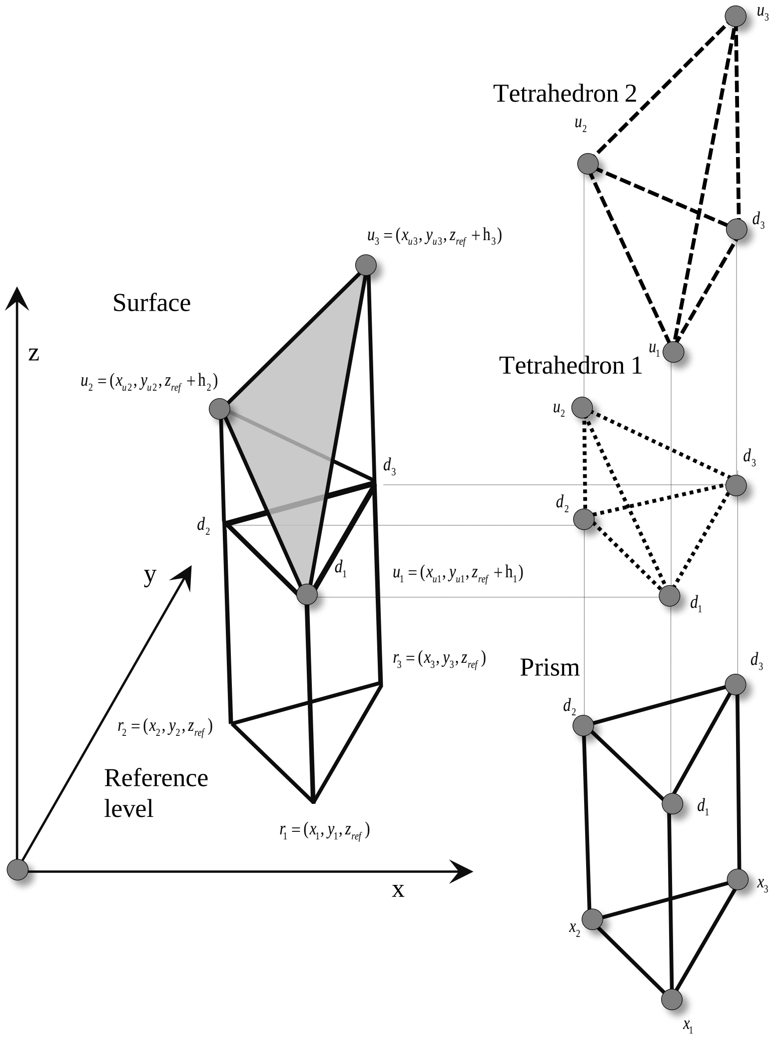


Figure 127: Volume calculation for TIN-data using triangle prism [Vulic, M. et al. 2006]

$$V_{Prisma} = BASE_{norm} \cdot h_1 \tag{52}$$

$$BASE_{norm} = \frac{\begin{vmatrix} x_{u1} & y_{u1} & 1 \\ x_{u2} & y_{u2} & 1 \\ x_{u3} & y_{u3} & 1 \end{vmatrix}}{2!} \tag{53}$$

At which the "$BASE_{norm}$" area of normal section, therefore triangles r_1, r_2 and r_3 or u_1, d_1 and d_3 (compare Figure 127). The volume of the first tetrahedron V_{T1}, which is formed by edges u_1, d_2, d_3, and u_2, can be calculated according to formula (54):

$$V_{(T1)u_1,d_2,d_3,u_2} = BASE_{norm} \cdot \frac{h_2 - h_1}{3} \tag{54}$$

Due to the fact that the volume of a pyramid does not change if the top of this pyramid moves on the surface which is paralleled to the basis. Founded on this principle, the volume of the second tetrahedron V_{T2} which is formed by edges u_1, u_2, d_3, and u_3, will not change if it parallels with the surface u_1, d_3 and u_3 move point u_2 to point d_2. The volume can be calculated with equation (55) [Vulic, M. et al. 2006]:

$$V_{(T2)u_1,d_2,d_3,u_2} = BASE_{norm} \cdot \frac{h_3 - h_1}{3} \tag{55}$$

If the volume of the regular prism and the two tetrahedrons are added, the total volume of the triangular prism can be obtained as a result using equations (56) and (57):

$$V_{Total} = V_{Prisma} + V_{T1} + V_{T2} = BASE_{norm} \cdot \frac{h_1}{2} + BASE_{norm} \cdot \frac{h_2 - h_1}{3} + BASE_{norm} \cdot \frac{h_3 - h_1}{3} \tag{56}$$

$$V_{Total} = BASE_{norm} \cdot \frac{h_1 + h_2 + h_3}{3} \tag{57}$$

Using this algorithm, both a volume bounded by a dredging polygon and a border line of a fairway with a certain width can be calculated. When this method is applied to the dredging polygons presented in Figure 121 and Figure 125 with a dredging depth ranging from 2.5 meters below LNWL to 3.0 meters below LNWL, surface of dredging volume for the shallow section Petronell-Witzelsdorf can be modelled. With increasing fairway width and depth, the required dredging volume is increasing, resulting in a concave rising dredging volume surface. If several smaller areas with insufficient depth exist within a shallow section, which are not connected in their shape, a dredging polygon can be created for each case followed by summing up individual volumes to a total dredging volume for this section.

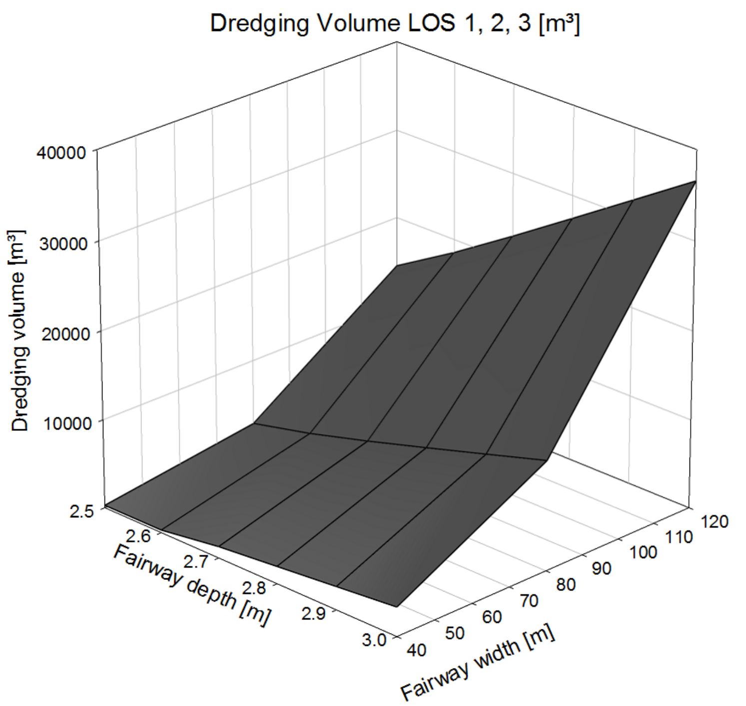


Figure 128: WAMS-example: necessary dredging volume (linear model visualization) for LOS1, LOS2, LOS3 and different target depths related to LNWL for ford Petronell-Witzelsdorf (TIN-survey data 22.10.2014) based on dredging polygons presented in section 5.5.2.

5.5.4 Cost estimation of dredging measures

Basic cost assumptions for the planning of dredging projects and the assessment of necessary budgets can either be based on the evaluation of already implemented dredging measures or may be determined as a first step by plausible expert guess. The following section provides an insight into the derivation of cost functions for dredging measures. In waterway asset management, flexible power functions, as introduced in chapter 2.3.6 (compare Figure 13), are used to describe the development of dredging costs. Already implemented dredging measures are stored in the WAMS-database, including measure extent, measure costs, type of material, dredging company and duration of measure implementation, and provide the basis for a determination of cost functions for selected projects using regression. As a result of the software technical realization, Figure 129 presents a cost function (58) for gravel also including confidence intervals and coefficient of determination that was derived based on a number of selected dredging measures that were implemented during a time period between 2011 and 2014. With an increasing number of already implemented measures being included in the database, the accuracy of cost estimations is increasing as well. With an increasing number of filtering criteria for the selection of appropriate measures for fitting of cost functions, planning of even very specific measures becomes feasible.

$$C_{T_{gravel}} = y = 0.769 + 78.916 \cdot x^{0.790} \tag{58}$$

Before the evaluation of dredging measures could be carried out based on a comprehensive WAMS-database, the long-term dredging statistics of VIADONAU were evaluated, as well, in order to determine performance indicators for dredging measures. The historical measures that were part of the statistical records were further imported into the WAMS-database. In the future, the old manual documentation will be replaced by a statistical evaluation function of the WAMS-software, and will also allow for automated evaluations of the performance of dredging measures. Unit costs of dredging measures may be calculated when the respective dredging costs of a project are divided by measures extent. As a result, unit cost functions for dredging measures can be obtained, representing economic scale effects.

Figure 130 provides such a unit cost function for gravel that is based on an evaluation of the statistics of dredging measures (VIADONAU: 2009-2013). As a result the average unit costs for gravel, considering the Austrian stretch of the Danube, amount to 8.36 €/m³.

Figure 131 presents a cost function (formula (59) for fine sediment also including confidence interval and coefficient of determination that was derived based on a number of selected dredging measures, that were implemented during a time period between 2011 and 2014.

$$C_{T_{fine\,sediment}} = y = -24.936 + 11.926 \cdot x^{0.880} \tag{59}$$

The development of unit cost for fine sediment dredging can be found in Figure 133 with average cost per unit amounting to 6,54 €/m³ . The evaluations clearly indicate that dredging of fine sediment is cheaper than dredging of gravel. As a major reason for this fact, particle shape and lower grain size of fine sediment particles may be identified, resulting in larger quantities of dredged material that can be loaded with the dredger to a single barge. Typical performances of dredgers will be shown in Figure 139 and Figure 140. Figure 141 will provide an overview of the most common dredging equipment.

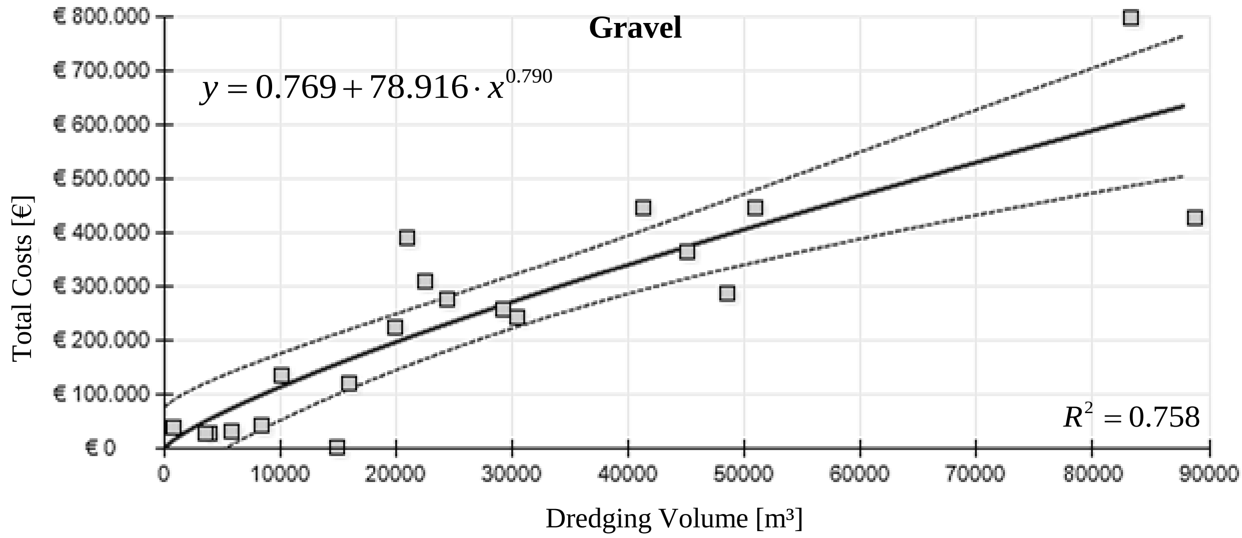


Figure 129: WAMS-example: cost function gravel based measures implemented between 2011-2014

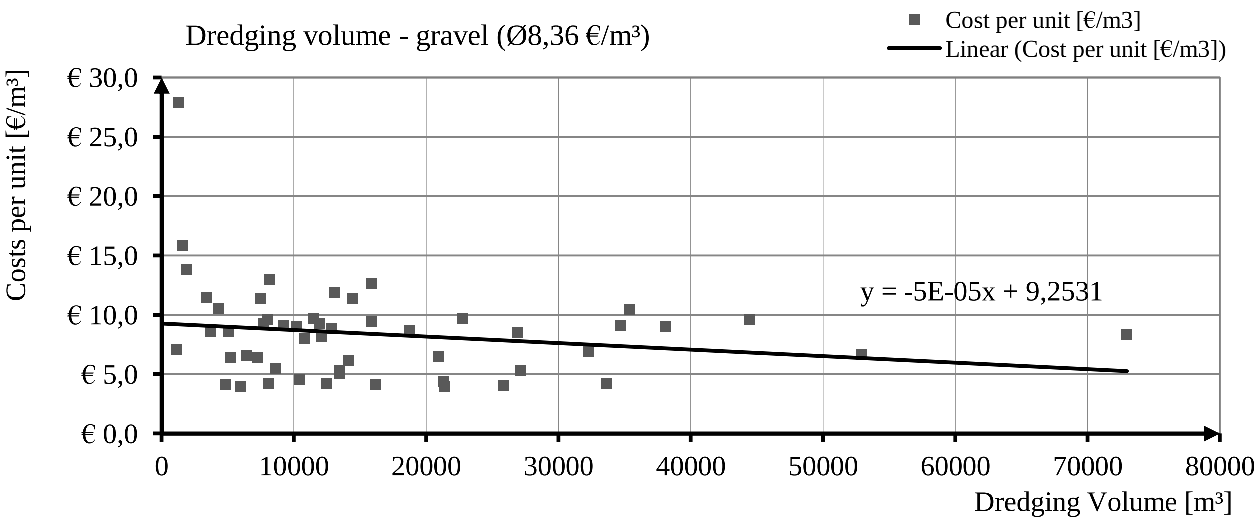


Figure 130: Costs per unit of gravel dredging depending on the extent of the measure on the Austrian stretch of the Danube for the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

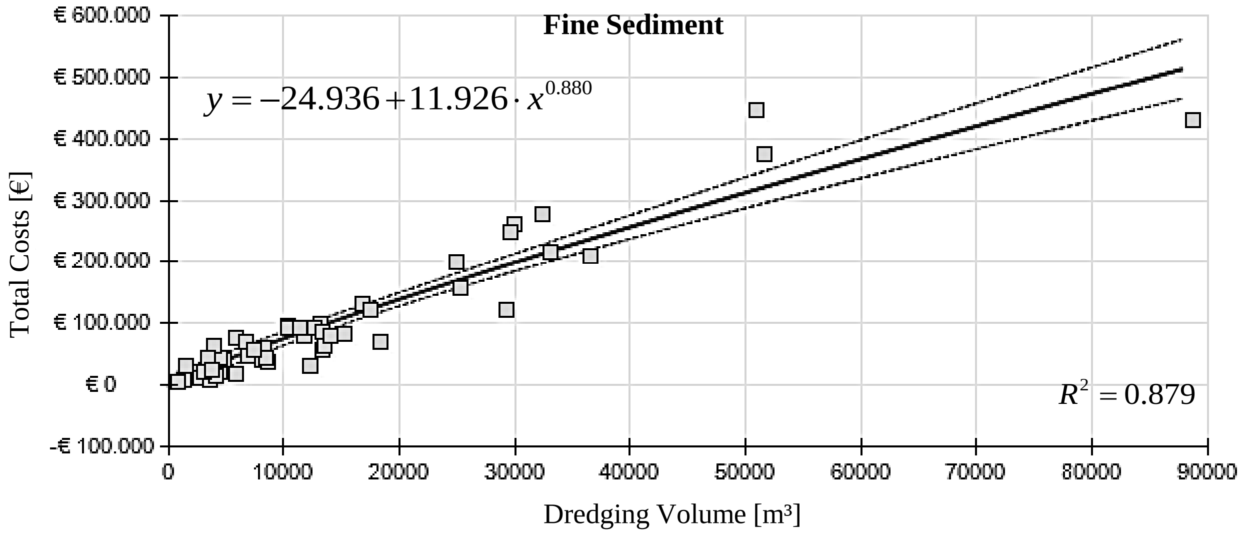


Figure 131: WAMS-example: cost function fine sediment based measures implemented between 2011-2014

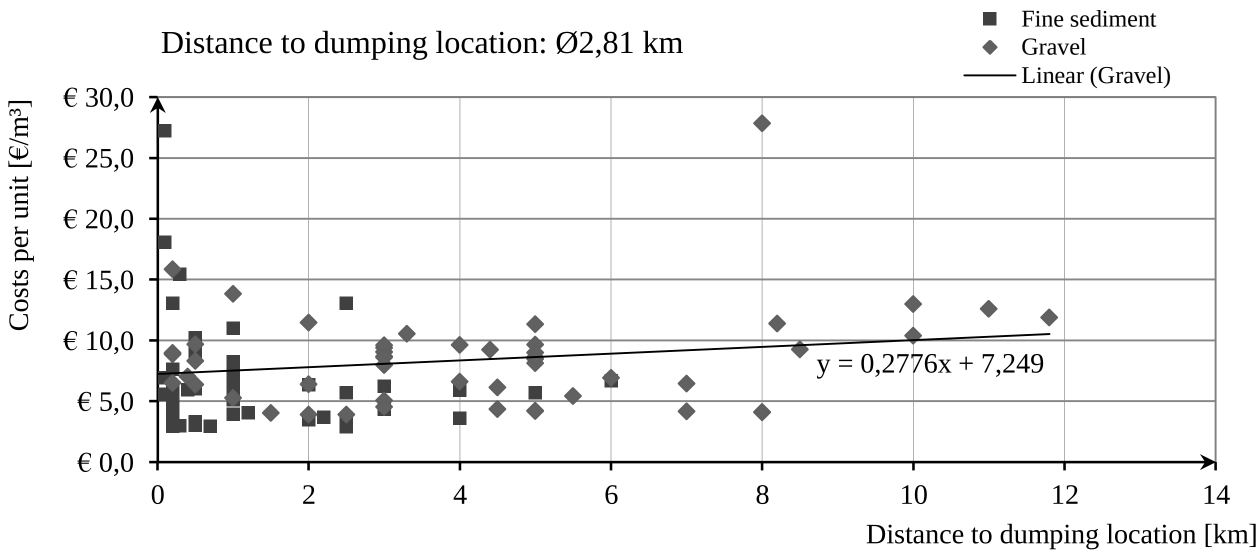


Figure 132: Costs per m³ dredging volume depending on the distance to dumping location on the Austrian stretch of the Danube for the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

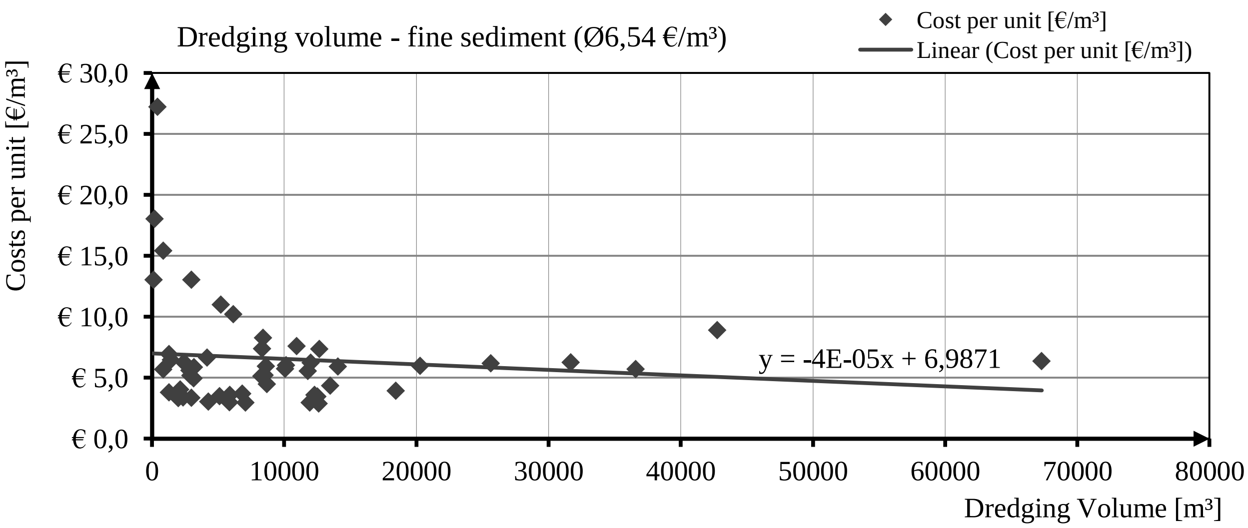


Figure 133: Costs per unit of fine sediment dredging depending on the extent of the measure on the Austrian stretch of the Danube for the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

After the dredged fine sediment of gravel is loaded on barges, it must be returned to the waterbody in order to alleviate inter alia deepening tendencies of the riverbed. For this purpose the material is usually transported upstream to a river section with high fairway depths and then dumped there. Further possible potential application possibilities of the dredging material include the construction of gravel structures for ecological purposes. Selling dredging material to the construction industry is not allowed due to environmental reasons as well as legal reasons. Figure 132 illustrates the development of unit costs with increasing distance to dumping location. The figure clearly shows that costs increase with increasing distance to dumping location. For the period between 2009 and 2012, the average distance to a dumping location was 2.81 km. In practice, fixed costs are used for transports within a certain category of distance such as 0-5 km, 5-10 km, 10-15km. As a result, a step-function of unit costs (as presented in Figure 26) is most suitable to describe the development of costs. Figure 134 present an analysis of dredging costs for ford Petronell-Witzelsdorf considering different dredging polygons (compare Figure 121, Figure 124 and Figure 125) that are related to defined levels of fairway width as well as dredging depths varying between 2.5 and 3.0 meters.

Each required dredging volume (Figure 128) corresponds to specific dredging costs that are calculated for gravel and fine sediment based on the cost functions given with formulas (58) and (59), and may also be visualized in the form of an increasing concave measure cost surface. In order to provide fairway with a width of 40m (LOS1) and a depth of 2.5 m below LNWL for this specific shallow section, only 7,149.49 € would be required. LOS1 at a fairway depth of 3.0 m below LNWL would result in dredging costs amounting to 153,065.70 €, whereas providing a fairway width of 120m (LOS3) at a fairway depth of 2.5 m below LNWL would already lead costs to 153,065.70 €. For a dredging depth of 3.0 m below LNWL, the necessary dredging costs are calculated at 318,103.30 €. The example of this shallow section shows, very clearly, that waterway operators have to consider carefully, which fairway width is actually required for current transport volumes.

Dredging costs depending on LOS & depth [€]

Fairway depth	**Fairway width**		
	40 m	**80 m**	**120 m**
2.5 m	7,149.49 €	41,746.36 €	153,065.70€
2.6 m	13,962.85 €	59,662.27 €	186,345.09 €
2.7 m	22,770.23 €	79,224.29 €	220,380.16€
2.8 m	32,001.61 €	98,504.66 €	253,795.78 €
2.9 m	40,563.98 €	116,823,74 €	286,783.44 €
3.0 m	47,532.73 €	135,268.56 €	318,103.30 €

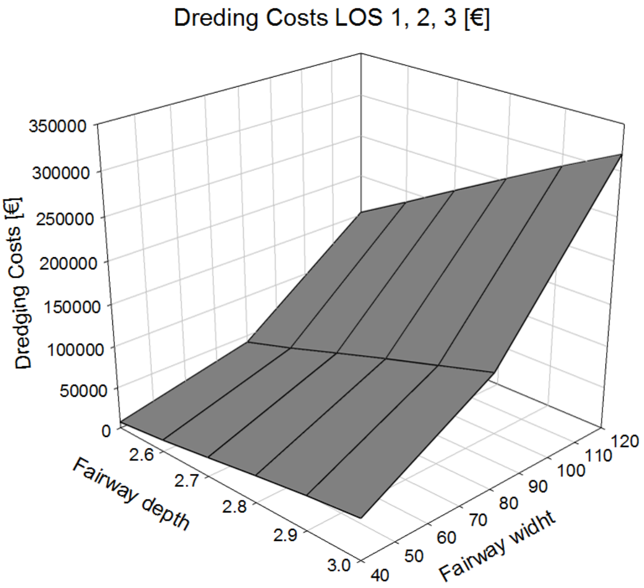
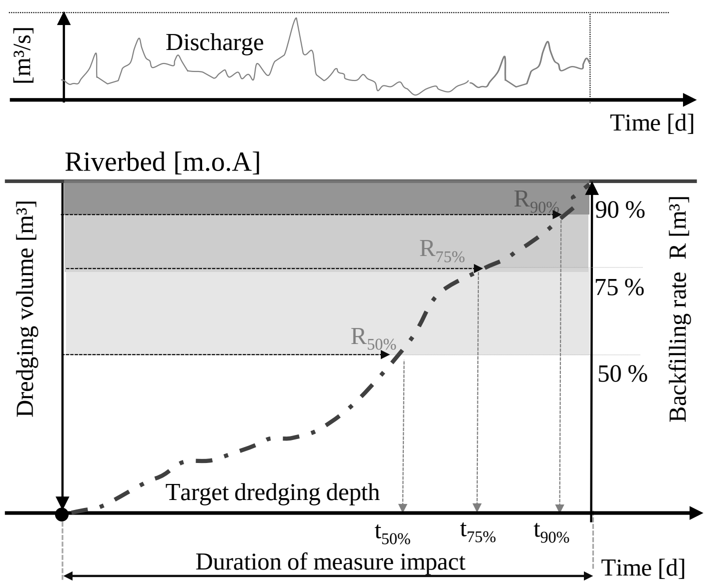


Figure 134: WAMS-example dredging cost surface (linear model visualization) for shallow section Pertronell-Witzelsdorf including total costs of dredging for the dredging polygons defined for LOS 1,2,3 and dredging depth of 2,5m to 3m below LNWL (TIN-data 22.10.2014)

5.5.5 Duration of measure impact

In general, the duration of measure impact ends if actual values for quality criteria (e.g. the availability of infrastructure) fall below predefined limits. The duration of impact of a dredging measure on a river section is calculated initially with the excavation of gravel or fine sediment cubature lasting until the total backfilling of the removed material, represented by the backfilling rate reaching 100% as shown in Figure 135. The duration of measure impact can be defined, as the time period in which an increased availability is given as compared to the zero alternative of "doing nothing". Thus, the measure with the highest impact on availability compared to necessary measure cost per time unit must be considered as favorable. Based on the analysis of historical data of water levels and riverbed surveys for a time period of 10 to 30 years, the characteristic development for each critical section can be derived as an empiric function of various impact parameters (e.g., discharge and structure of riverbed material). The schematic gradient of riverbed development, e.g. above Adriatic Sea, and the backfilling rate of the excavated volume can be derived on the basis of analysis results from one comprehensive database depending on the progressivity of the respective erosion and deposition curves. A high progressivity and frequency of interventions indicate that dredging measures are not appropriate for this section and more lasting measures, such as river engineering works should be considered [Haselbauer K. et al. 2014].

Figure 135: Duration of a dredging measure based on the typical backfilling rate of the dredged material related to the discharge in the time period [Haselbauer K. et al. 2014]



The characteristics of river sections which show a flatter schematic gradient of riverbed development over time, as compared to the average can be classified as relatively stable. By using defined safety levels below the bottom of the fairway (parallel lines to the water level), critical developments of the riverbed can be automatically identified, acting as an innovative alert system (Figure 135). The highest priority should always be applied to the critical section showing the lowest fairway depth on the entire width of the fairway. Since this kind of sedimentation in the fairway leaves no room for bypassing, such a section is considered as highly critical for navigation companies in the case of low-water levels. Second in the ranking system are critical sections which are characterized by low fairway depths only at the limits of the fairway. Considering long-term data sets, the backfilling function of each river section will show a different characteristic behavior, which can be derived transversely, for example, with impact parameters such as the predominant discharge.

For a certain critical section, the optimal timing of a dredging measure may be determined by a variation of excavation time and subsequent evaluation of the backfilling behavior. Thus, the intervention time with the flattest gradient of the backfilling rate will result in the highest measure impact. Due to environmental reasons, the maximum target dredging depth within the fairway in Austria in free-flowing sections of the Danube is restricted to 2.5 m + 0.5 m tolerance below low navigable water level (LNWL). If the duration of the measure impact is considered together with measure costs the resulting annual costs, can be calculated based on standard equation (25) described in chapter 2.3.14.

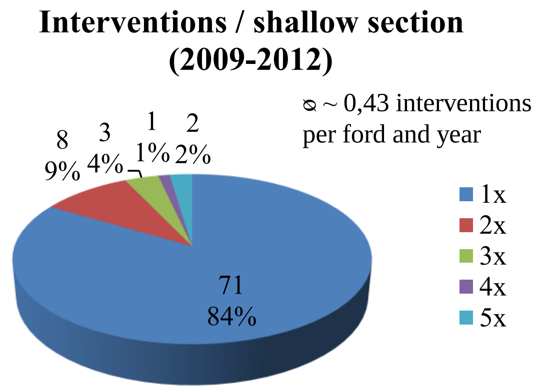


Figure 136: Intervention frequency for shallow section at the river Danube during the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

Figure 136 provides an overview of the analysis of statistics of dredging measures. For 84% of listed shallow sections, one intervention within 3 years was required. Thus, on average, 2.32 dredging measures per shallow section were implemented during the years 2009 and 2012. According to Figure 137, the highest amount of measures was thereby implemented during the months January (29), February (31) and March (30). Legal restrictions of dredging measures due to spawning periods excluding emergency interventions result in a low number of dredging measures being performed in spring months.

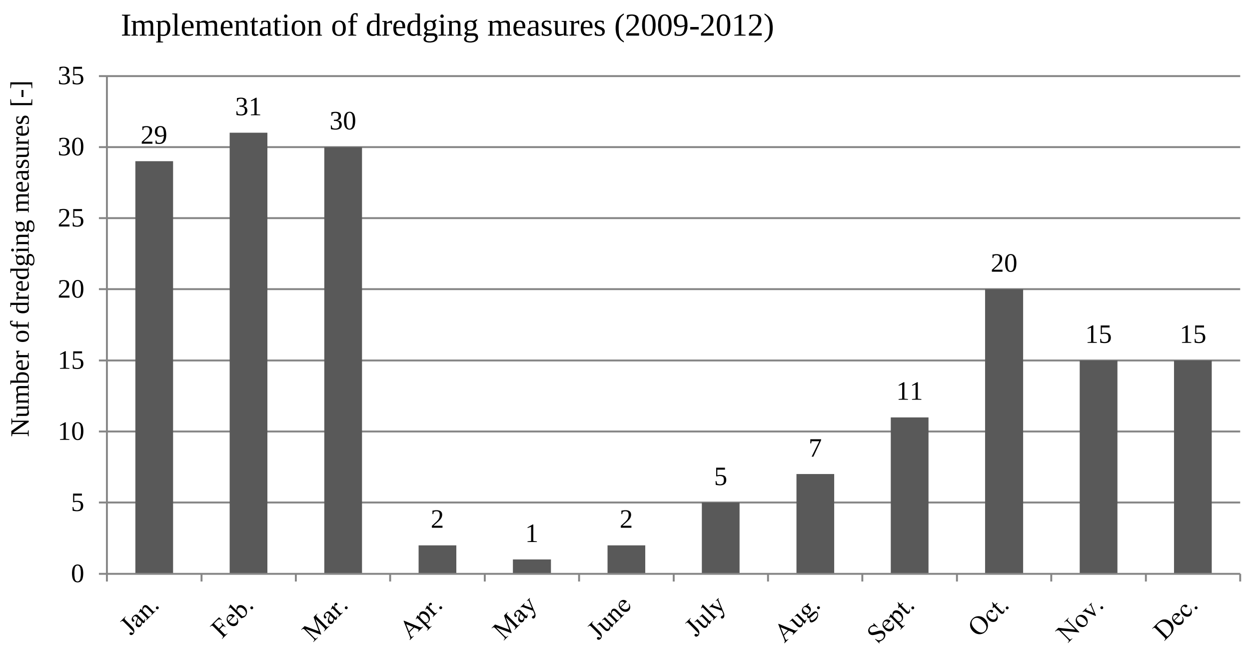


Figure 137: Number of dredging measures implemented on the Austrian stretch of the river Danube for the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

In practice, the total measure extent per year as well as any individual measure extent and time to implementation are limited by technical, environmental and economic reasons (e.g. available number and quality of dredging equipment on the market, temporal restrictions regarding maintenance interventions, annual budget of waterway authorities). The presented approach provides the means to achieve unified and continuous fairway availability levels and may lead to an efficient allocation of available budget or funds. As part of a holistic WAMS, such an approach may facilitate the ranking of intervention times and a prioritization of existing critical sections.

5.5.6 Implementation of dredging measures – dredging performance

Maintenance dredging is defined as removal of sediments and debris from the bottom of rivers, lakes, ports, and other water bodies. It is a routine necessity on waterways in the fairway because natural sedimentation processes are gradually filling fairway channels and port entrances. Dredging is often focused on maintaining or increasing the depth of the fairway for a given width to ensure the safe passage of vessels without touching the ground. Vessels require a certain amount of water in order to float and avoid groundings. The necessary fairway depth is a function of static draught, dynamic squat and Underkeel clearance. With dynamic squat being mainly related to vessel speed and a minimum Underkeel clearance to prevent groundings, the remaining static draught depends on vessel utilization. International recommendations for certain fairway classes define fairway availability as a function of width and depth in days per year. Since utilization of the entire fleet is mainly related and limited to minimum depth on an entire transport route, dredging as a fast and effective measure plays a vital role in providing competitive conditions for inland navigation.

Apart from analysis and optimization options in full waterway asset management current typical planning processes for dredging measures consist of an assessment of most critical sections regarding the available fairway depth for a certain width (Levels of Service – LOS). For these sections, dredging plans are designed with an additional allowance for the design depth in order to account for local backfilling during the measure as well as for equipment-related inaccuracy in the performance of the dredging work itself. As a result, a certain volume can be calculated that has to be dredged in a given area with the material normally being dumped upstream (e.g. with hopper barges) in order to avoid riverbed erosion or being used for other purposes (building islands, use as construction material). Depending on the situation and the type of dredged material, different dredging equipment will be appropriate. Figure 141 provides an overview of dredgers and transport equipment, along with engine power, transport capacity and maximum dredging depth being in world-wide use. Most common equipment on the waterway Danube, for dredging mainly in critical sections and port areas, are backhoe dredgers and cutter suction dredgers (Figure 138). Dredging costs for a given dredging volume, e.g. on shallow sections, depend mainly on dredger performance, labor costs, dredging acquisition and maintenance costs, fuel and transport distance to dumping site, and have a decreasing tendency of dredging unit costs with increasing dredging volume (economy of scale). Prices, on the other hand depend on the situation of whether agencies are dredging by themselves or are tendering dredging works on an only partially functioning market with limited capacity [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Common backhoe dredger on the upper Danube performing dredging works in Austria for viadonau

Common cutter suction dredger on the central Danube owned since 2001 by Plovput in Serbia

Figure 138: Examples for commonly used dredging equipment on the Danube waterway [Hoffmann, M. et al. 2014]

The duration of measure implementation in general depends on the amount of dredging volume and performance of the dredging equipment, with total dredging time being a result of dredging volume divided by dredging performance in days. In addition, there are some restrictions as to the use of certain dredging equipment, such as maximum draught and flow velocity, that have to be considered. In low-water periods, hopper barges cannot be fully loaded, which substantially increases the number of required trips to transport the same amount of dredging volume as well as the resulting costs. Important for planning of dredging activities is the timely rehabilitation of shallow sections before a probable occurrence of low water periods as these might form obstacles for inland navigation. Therefore, it is necessary to know the average daily performance of excavators in order to estimate how many dredging days are needed and how many excavators have to be used simultaneously.

Figure 139 provides an overview of the dredging performance for fine sediment using standard backhoe dredgers on the Austrian Danube stretch. Thus, the average dredging performance for fine sediment is 1.829 [m³/d]. The dredging performance for gravel is illustrated in Figure 140 and indicates a significantly lower dredging performance per day. Thus, common backhoe dredgers offer a daily dredging performance of 1201 m³. This difference may be due to a lower bulk density of gravel compared to fine sediment.

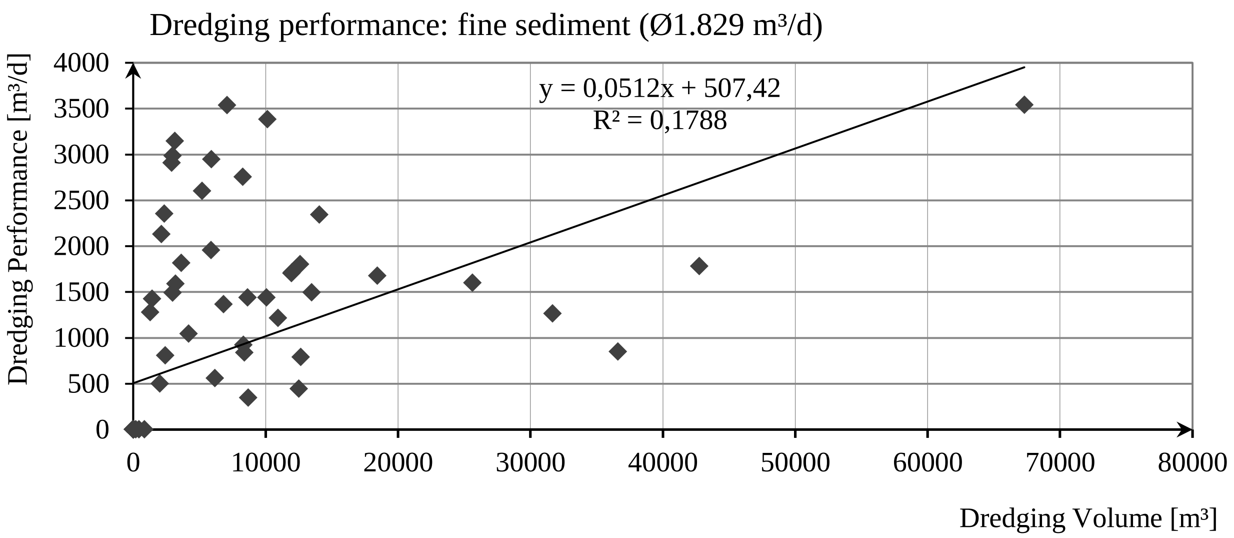


Figure 139: Dredging performance for fine sediment on the Austrian stretch of the Danube for the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

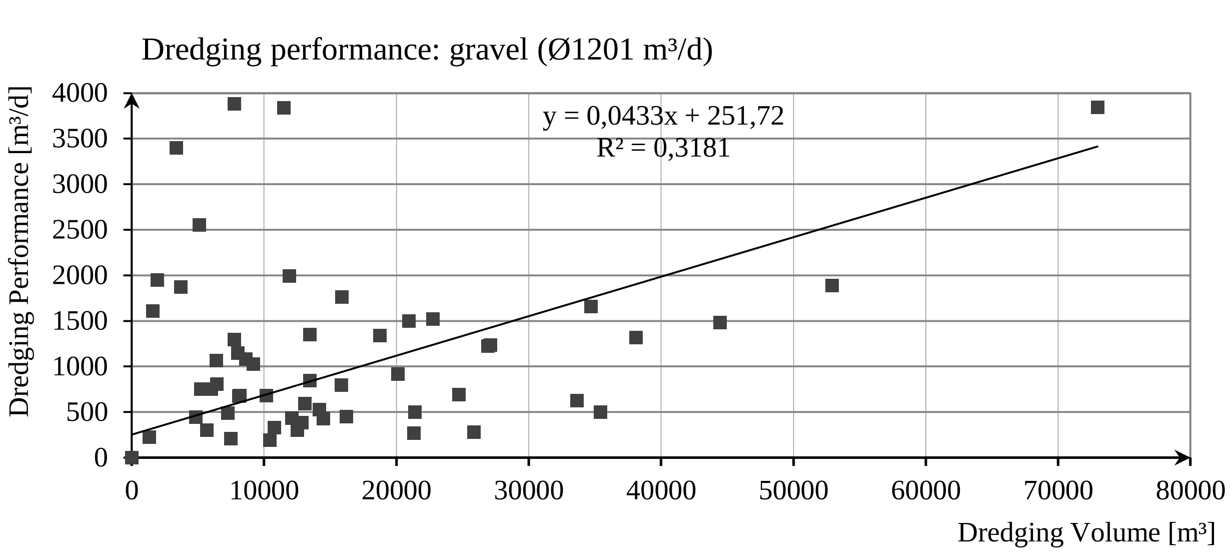
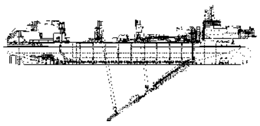


Figure 140: Dredging performance for fine sediment on the Austrian stretch of the Danube for the years 2009 to 2013 [VIADONAU, statistics of dredging measures]

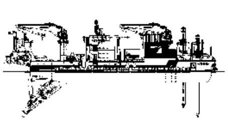
Dredging of waterways, shallow sections and harbour areas – selected examples of typical dredging approaches and equipment being available on the market

Type: Trailer suction hopper dredger




- Large, powerful pumps and engines that enable it to suck up sand, clay, sludge and even gravel from ocean or river beds
- Discharge of material by dumping, pump – pressing, rainbowing and craning
- Power: 1.500 - 30.000 kW, Hopper capacity: 1.000 – 40.000 m^3 Max. dredging depth: 20 – 100 m

Type: Cutter suction dredger

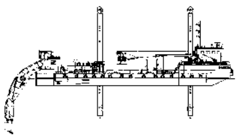



- stationary or self-propelled vessel that uses a rotating cutter head to loosen the material in the bed (‘cutting’)
- Discharge of dredged material directly to shore via a floating pipeline or into a barge
- Power: 400 – 25.000 kW, Discharge pipe: 250 – 1.000 mm Max. dredging depth: 9 – 30 m

Type: Suction dredgers

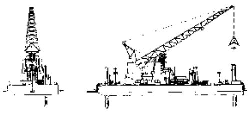

- stationary or self-propelled vessel that uses sucking through a long tube, like vacuum cleaners of sand, clay or sludge
- Discharge of dredged material directly to shore via a floating pipeline or into a barge
- Power: 500 - 10.000 kW, Discharge pipe: 250 – 1.000 mm Max. dredging depth: 20 – 60 m

Type: Backhoe dredgers



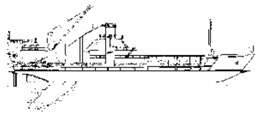

- Hydraulic grab crane on a dredging pontoon held in place by three spud poles to dredge heavy clay, soft stone, blast rock
- Discharge of dredged material into a hopper or pushed barges
- Power: 500 - 5.000 kW, Grab capacity: 1,5 – 25 m^3 Max. dredging depth: 5 to 30 m

Type: Clamshell dredgers



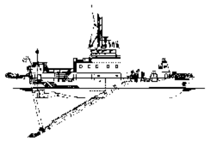

- A grab dredger with a clam shell bucket from an onboard crane or a crane barge for excavation of soft clay, sand and gravel
- Discharge of dredged material into a hopper or pushed barges
- Power: 200 - 2.000 kW, Grab capacity: 1 – 20 m^3 Max. dredging depth: 5 to 100 m

Type: Bucket (ladder) dredges



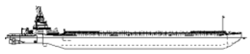

- The inclined bucket ladder rotates as endless chain with sand/gravel beeing scooped and discharged at the upper end
- Discharge of material into the dredge hold or a barge
- Positioning: anchoring, mooring winches
- Power chain: 40-200 kW, Capacity: 200 – 1.000 t/h Max. dredging depth: 5 – 25 m

Type: Water injection dredges




- An injection beam located underneath the vessel injects large volumes of water under low pressure into the sediment
- The suspended silt and fine sand turns into a density current, being removed by of gravity/current
- These dredgers are mainly used in small, shallow areas
- Power: 50 – 2.500 kW, Max. dredging depth: 5 – 20 m

Type: Hopper barges

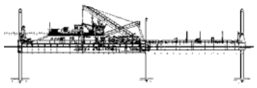



- For self propelled transport of large amounts of excavation material to dumping/discharge site
- Power: 150-500 KW, Capacity: 2.000 – 10.000 m^3

Type: Split hopper barges

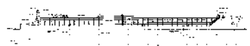

- For self propelled transport of excavation material to dumping/discharge site
- Power: 50 - 4.000 KW, Capacity: 500 – 3.000 m^3

Type: Barge unloading dredgers




- For transport of excavation to dumping/discharge site
- Power: 1.000 - 5.000 KW, Pipe: 250 – 1.000 mm

Type: Pushed barges




- For transport of excavation to dumping/discharge site
- Power: Pushed Capacity: 800 – 2.000 m^3

Figure 141: Selected examples of typical dredger and transport of excavated material being in use on waterways, critical sections, ports or marine channels [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].]

5.5.7 Harmonized fairway depth

The presented approach includes an identification of critical sections, such as fords and narrow sections, using an algorithm for an automated linkage of neighboring critical sectors, e.g. based on single-beam surveys, to one shallow section, thus providing the basis for a real-time monitoring system with a current catalogue of critical sectors in the background. The resulting total availability of a transport route is determined by the shallowest and/or narrowest section. Therefore, the minimal fairway depth on a transport route determines the possible utilization of the vessel fleet. A lower availability of fairway width, e.g. with one-way traffic on several sections, results in an accumulation of waiting times in a series of narrow sections. Currently, transport on the Danube utilizes only a fraction of its transport capacity. Thus, possible encounters of convoys with the most critical dimensions (4% of the fleet on the upper Danube) are very rare, making it unlikely to occur very often on a transport route. Compared to possible transport durations of a few weeks, possible time and transport cost savings can be considered as almost negligible. Thus, dredging the entire width of the fairway with resulting high costs of necessary measures will not be cost-effective.

The serial model of section availability regarding fairway depth is limited by the innermost availability performance of the most critical section on a transport route in order to provide continuous navigation conditions. Improved continuous fairway conditions can be achieved by shifting (e.g. dredging of the riverbed) the availability performance of the most critical section beyond predefined fairway target parameters (e.g. DC 2013 for the Austrian section of the Danube: width = minimum 100/120 m and maximum 120/150 m, vessel draught = 2.5 m). For a serial system, the effectiveness of measures on availability is limited depending on the condition of the next most critical section. Further expenditures towards improving the availability on the first critical section are therefore a waste of budget if the conditions on the other critical sections are not improved up to a certain common target level (Figure 142).

For a manual maintenance optimization, the availability curves may be shifted outward until the budget is spent. For implementation purposes, the respective functions of riverbed development, water level forecasts and performance of used equipment have to be included as well. An automated measure planning mode in a WAMS is be able to outline the required dredging volume, necessary sequence of measures and resulting financial requirements for each combination of fairway width and depth as a basis for optimized investment decisions and determination of annual budgeting needs [Haselbauer, K. et al. 2014].

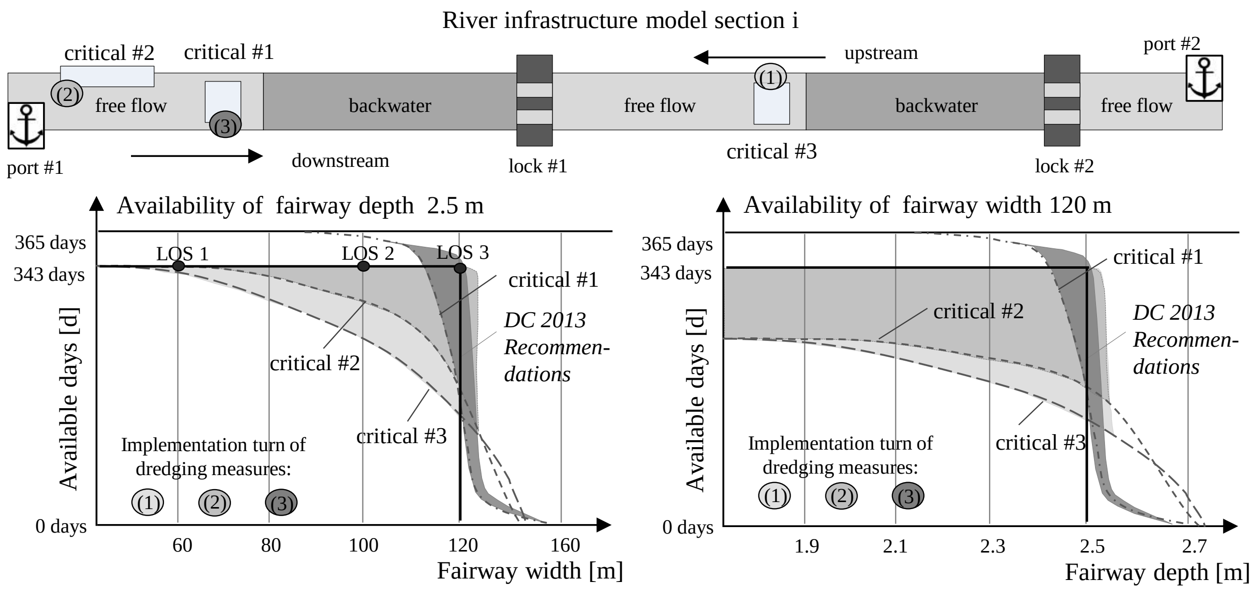


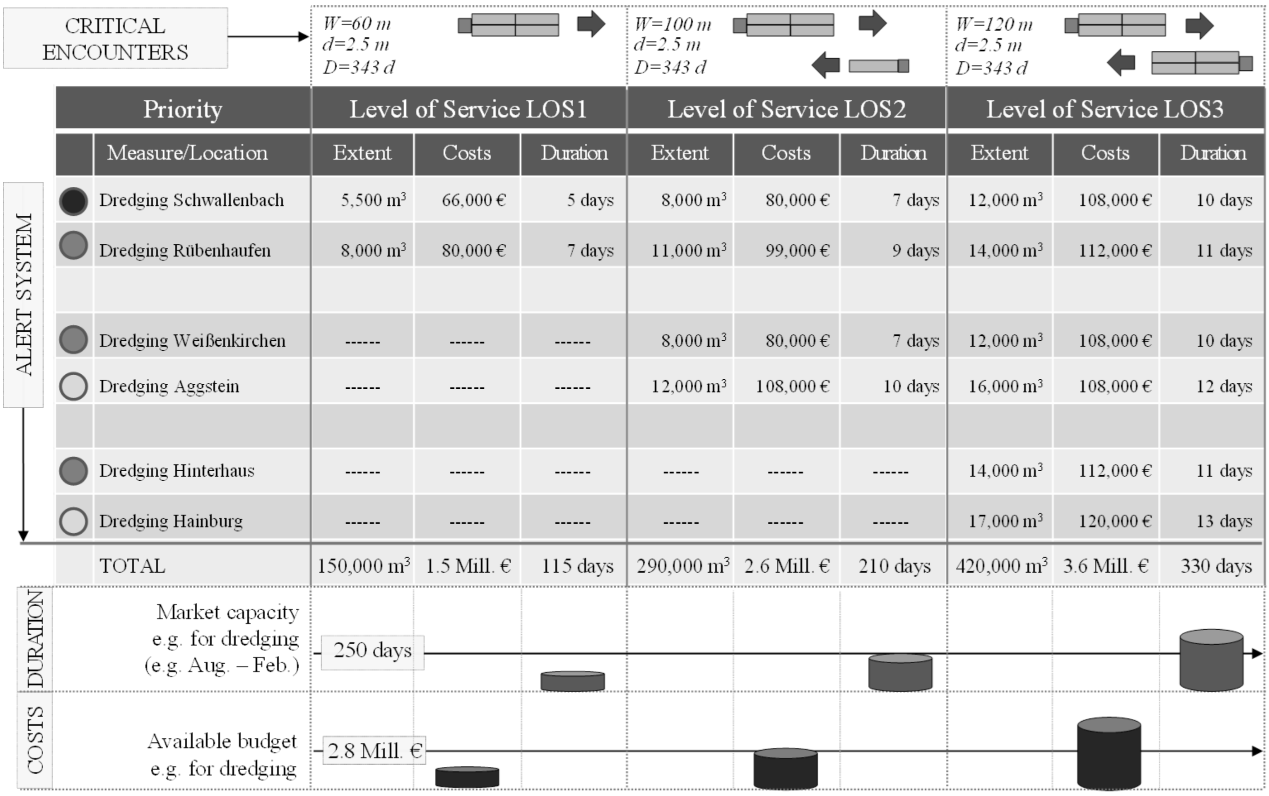
Figure 142: Optimization of fairway parameters and resulting availability on a transport route with different critical sections.

5.5.8 Measure program

As a result of this optimization process, all resulting measures aiming at a certain Level of Service (LOS) may be condensed into a measure program containing priority, measure extent and costs as well as time for implementation. Figure 143 shows a conceptual overview of such a program for different LOS with a main emphasis on, but not limited to, dredging measures.

On the most critical sections regarding availability, certain measures (e.g. dredging) will be necessary even for lower requirements regarding fairway parameters (e.g. LOS1). With higher requirements regarding LOS the number of critical sections with necessary measures will increase. The same holds true for necessary measure extent as well as resulting costs and time for implementation. The priority between measures for achieving a certain Level of Service is based on the critical behavior of individual sections (alert system). The highest priority is given to shallow sections with a very low fairway depth within the central fairway area. The respective available fairway depth below LNWL is therefore decisive. Thus, possible negative impacts on inland navigation can be minimized with implementation in time prior to arriving at critical conditions. All measures to achieve a certain LOS result in a total measure extent, cost and time.

The implementation of such a resulting measure program may be limited by several factors. In Austria, the timeframe for possible interventions in the riverbed is restricted to certain periods due to environmental reasons (e.g. spawning season of fish). Further restrictions and shifts in priorities may apply due to appearing low water periods. For dredging as a main measure in Austria, the possible dredging volume may also be limited due to the current market capacity of available dredging equipment. Even with a sufficient budget at hand, it would still not be possible in certain cases to implement all measures in time to achieve targeted fairway availability and minimize negative impacts on inland navigation without setting priorities and/or limiting fairway width [Hoffmann, M. & Haselbauer, K. & Blab, R. & Simoner, M. & Dieplinger, K. & Hartl, T. 2014a].



Priority	Level of Service LOS1			Level of Service LOS2			Level of Service LOS3		
Measure/Location	Extent	Costs	Duration	Extent	Costs	Duration	Extent	Costs	Duration
Dredging Schwallenbach	5,500 m³	66,000 €	5 days	8,000 m³	80,000 €	7 days	12,000 m³	108,000 €	10 days
Dredging Rübenhaufen	8,000 m³	80,000 €	7 days	11,000 m³	99,000 €	9 days	14,000 m³	112,000 €	11 days
Dredging Weißenkirchen	------	------	------	8,000 m³	80,000 €	7 days	12,000 m³	108,000 €	10 days
Dredging Aggstein	------	------	------	12,000 m³	108,000 €	10 days	16,000 m³	108,000 €	12 days
Dredging Hinterhaus	------	------	------	------	------	------	14,000 m³	112,000 €	11 days
Dredging Hainburg	------	------	------	------	------	------	17,000 m³	120,000 €	13 days
TOTAL	150,000 m³	1.5 Mill. €	115 days	290,000 m³	2.6 Mill. €	210 days	420,000 m³	3.6 Mill. €	330 days

Figure 143: Resulting measure program e.g. for dredging measures with measure extent, measure costs and time for implementation depending on targeted LOS and priority (e.g. due to critical condition or development)

6 TRANSPORT COSTS AND EFFICIENCY

6.1 Fairway availability and resulting transport costs

Due to the linear structure of waterways, one single bottleneck with insufficient loading depth will limit the utilization of the entire transport fleet. For linear transport modes with a serial structure, it is therefore particularly important that fairway depths are continuously available on the entire transport route. For typical goods and vessel types on the Danube, 1.0 cm of additional loading depth results in 7 to 14 t of further goods capacity. With variable loading depths leading to an average utilization of 55 to 60% on the upper Danube and 50 to 55% on the lower Danube, inland navigation was not able to use its potential. The transported freight volumes are presented in Figure 144.

For an assessment of whether the supplied infrastructure availability meets the needs of the users of the waterway, it is essential for waterway authorities to have an overview of transported cargo volumes during the year, including the composition of the cargo vessel fleet navigating on the national river stretch. The transport volume on the Austrian section of the Danube, for example, shows a steady tendency with around one million passengers and nine to eleven million tons of goods transported per year.

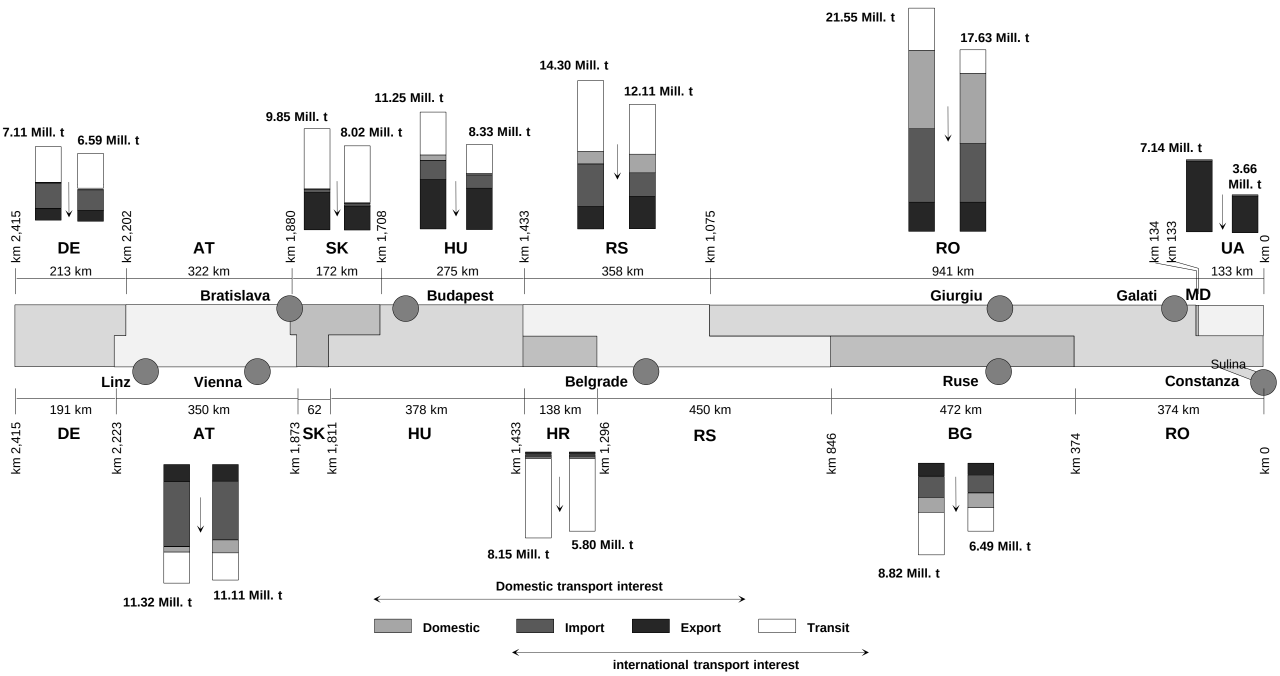


Figure 144: Overview freight transport volumes on the Danube in 2010 and 2012 [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

In 2012, the total number of vessels locked through the Altenwörth river hydropower plant amounted to 10,700 units with 35% being passenger vessels and 65% cargo vessels and convoys. Passenger transport shows a steady high season between April and October whereas cargo transport shows more fluctuations depending on the prevailing water levels and market conditions of the types of goods transported (Figure 145). The average load factor for cargo vessels on the Austrian section of the Danube usually ranges from 60 to 68% but may drop to 40% in severe low-water periods (as for example in the summer of 2003). On the lower Danube, the average load factor varies between 50 to 55%. As shown in Table 4, which provides an overview of the throughput of vessels at the Austrian Melk and Altenwörth locks in 2012, the majority or 51.6% of journeys on the upper Danube are performed by individual vessels consisting mainly of the vessel types Johann Welker or extended Gustav Koenigs [Hoffmann, M. et al. 2014b].

The second most frequent types are a combination of a pusher and two barges with a fraction of 26.5%. Another frequent type, with 11.5%, is large motor cargo vessels in combination with one barge. Furthermore, the provided table allows the calculation of critical encounter probabilities between cargo vessels and convoys making a determination of waiting times and costs at narrow river sections possible as well.

The analysis of the cargo vessel fleet allows for further insights into the possible and actual utilization of the physical availability of the waterway. The fleet on the upper Danube mainly consists of self-propelled motor cargo vessels and pushed convoys consisting of a pusher and one to four barges. On the lower Danube, pushed convoys with a pusher and up to nine or more barges are used.

Vessel/convoy type	Melk 2012	Altenwörth 2012	share [%]
Single vessels	3,735	3,507	51.6%
Coupled convoy 1xbarge	824	796	11.5%
Coupled convoy 2xbarge	34	31	0.5%
Coupled convoy 3xBarge	23	22	0.3%
Pushed convoy 1xbarge	346	252	4.3%
Pushed convoy 2xbarge	1,762	1,958	26.5%
Pushed convoy 3xbarge	75	91	1.2%
Pushed convoy 4xbarge	293	297	4.2%
Total number vessels	**7,092**	**6,954**	**100.0%**

Table 4: Cargo fleet composition on the upper Danube based on locked-through vessels at Melk and Altenwörth in 2012.

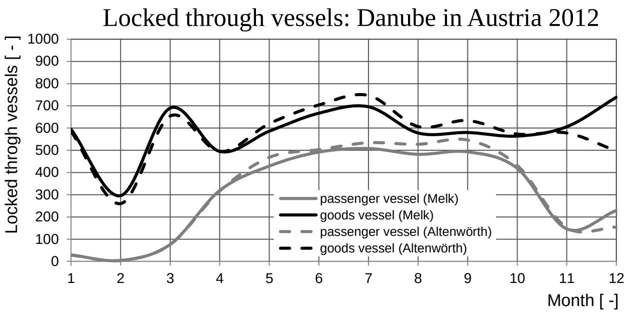


Figure 145: Locked-through vessels per month at the hydroelectric power stations in Melk and Altenwörth in Austria 2012

With a share of merely 4.2% for four-unit pushed convoys (pusher with four barges) and an average frequency of 19 goods vessels per day (8.5 per direction/day), the critical encounter probability for LOS 3 (oncoming traffic with two four-unit pushed convoys passing) is less than once a week and even lower in the few critical sections, which amount to a small fraction of the entire transport route (Figure 146) with just a few minutes of waiting time in the worst case. Therefore, fairway widths will not be an issue even with a possible future substantial increase in transport volumes on the Danube [Hoffmann, M. et al. 2014b].

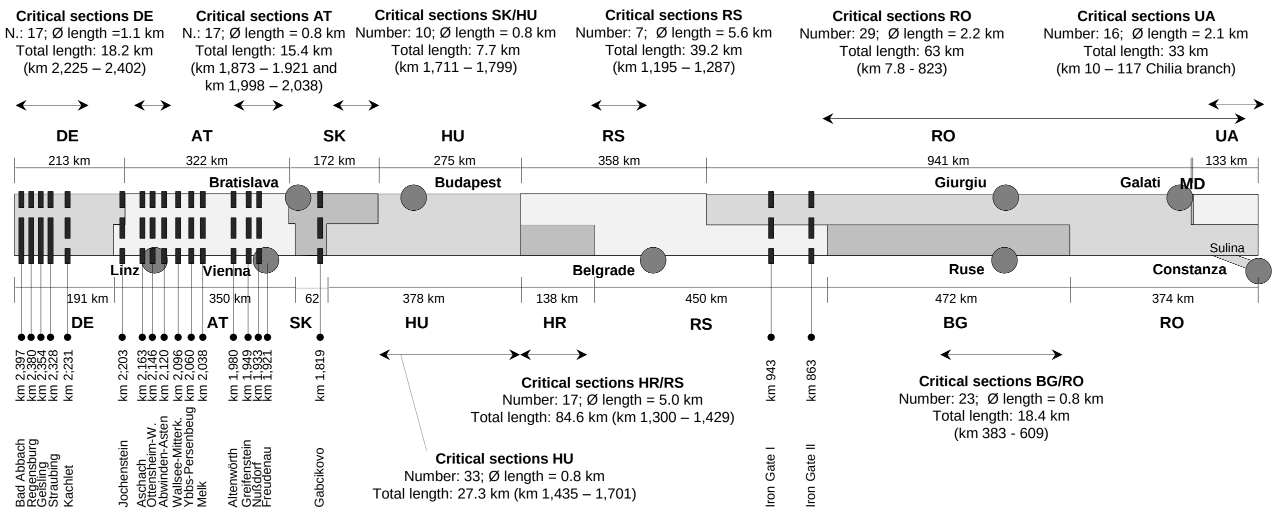


Figure 146: Overview of nautical bottlenecks: hydro power plants with locks and critical sections [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]

6.2 Availability of fairway depth and possible vessel utilization

For the attractiveness of inland waterways and the competitiveness of navigation companies in the transport market, a high utilization of transport capacity is essential throughout the year. The available fairway depth on any given day determines the amount of goods that may be carried on an inland cargo vessel. The fairway depth needed for a trip of an individual vessel consists of static vessel draught (draught loaded), dynamic squat and an underkeel clearance, and must be lower than the actual available fairway depth. Depending on vessel type, calibration curves link possible utilization and loaded draught static (velocity = 0). Figure 147 (a) provides an overview of draught loaded (static draught) for the most common vessel types on the Danube. For example, the most common single vessel type (Johann Welker) provides a static draught loaded of 2.5 m at a load factor of around 96%. A pushed convoy with two barges and the same draught loaded shows a load factor of only 54%. If draught loaded drops below 2.0 m, utilization decreases to 64% for the single vessel type Johann Welker and to 38% for typical pushed convoys with two barges [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

Depending on vessel speed in shallow waters an area of lowered pressure can be formed causing the ship to dive into the water. This dynamic squat depends on vessel speed, among other factors, and ranges from 0.05 to 0.5 m for the afore-mentioned vessel types and for a speed between 5 and 15 km/h (Figure 147 (b)). Experienced captains therefore decrease vessel speed at already known or properly marked critical sections, or increase vessel speed in order to clear bridges during high water levels. In order to prevent groundings or damage to the propulsion system of vessels, in addition to static draught and dynamic squat, the underkeel clearance has to be considered as well. According to VIADONAU [2013b], the underkeel clearance is at least 0.2 m for gravel and 0.3 m for rock on the riverbed. Even though these factors and their impact on necessary fairway depths are well known, the main uncertainty lies in getting an accurate estimation of water levels prior to loading and knowledge of actual conditions upon arrival at shallow sections. Therefore, the physically available fairway depth is almost never fully utilized in practice. Though permanent riverbed surveys are not possible, the actual information from echo sounders of already passed ships could be made available for all customers. If this information would be validated in a systematic way, this would surely improve the relevance and reliability of provided information on fairway availability [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

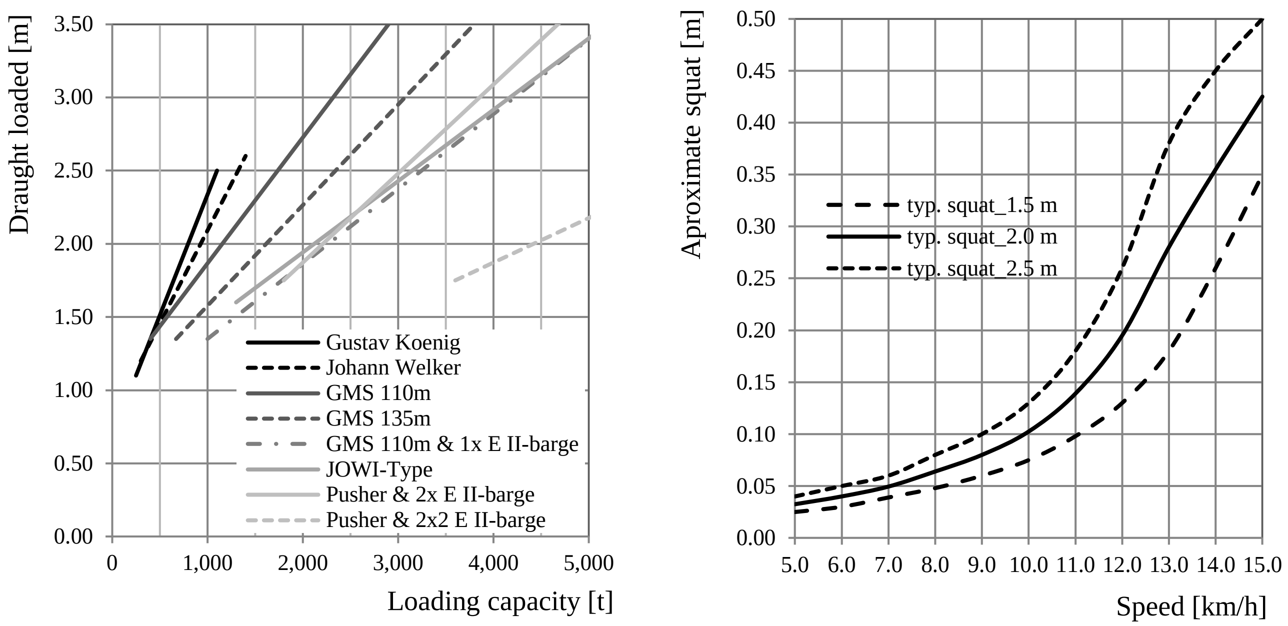


Figure 147: (a) Draught loaded (static draught) and loading capacity of river Danube fleet, (b)Dynamic squat depending on draught loaded and speed for low water levels and typical ships of the Danube fleet [Hoffmann, M. et al. 2014a]

6.3 Potentials and risks related to fairway conditions

With the number of different shallow or narrow sections at low water levels on typical long transport distances being relatively high, the probability of one section being critical is also very high. Due to the linear structure of this mode of transport, only one remaining shallow section is enough to limit the draught loaded of the entire vessel fleet on this transport route. Compared to relatively long transport times between one to three weeks, the impacts on transport costs of a few minutes waiting time at narrow sections are negligible if captains of encountering vessels are coordinated properly. Thus, restrictions of fairway width on transport routes with low transport volumes have almost no impact on transport costs. If the average load factor of vessels drops to 50% or more due to insufficient fairway depths, this mode will not be competitive when compared to subsidized rail transport, according to market analysis and interviews. If it is possible to increase the load factor of vessels up to 70% due to improved fairway conditions, actual information and transport logistics, it will be hard for road and rail to compete in the goods market for transport distances exceeding 500 to 800 km in the Danube corridor. Such a stable situation could also encourage much-needed investments in ageing waterway infrastructure, vessel fleets and equipment [Hoffmann, M. et al 2014b].

6.4 Studies on road, rail and waterway transport costs

Reliable and available transport infrastructures as well as resulting transport costs are of crucial importance with regard to the competitiveness of different modes of transport on the market. Depending on the type and amount of goods, transport relations, transport distance and possible utilization, the choice for a mode of transport or intermodal transport chain will be different. For shippers and logistics service providers, the ratio of price and performance mostly determines the individual case by-case decision for a mode of transport. In practice, the total transport duration, including unloaded journeys as well as loading and unloading times, is calculated in the first step. In general, transport cost models therefore consist of variable time-dependent, transport distance-related (operating costs) and fixed cost components (standby costs). Standby costs include crew wages, maintenance and repairs, amortization of vessels and insurance. Operating costs include bunker and lubricant costs, commission for brokering contracts, dues, fees and fuel consumption. Furthermore, utilization and transport relation have a major impact on the resulting transport unit costs. Depending on the complexity and implemented cost components almost all transport cost models show a convex decreasing form with increasing transport distance [Hoffmann, M. et al. 2014b]. Hanssen et al. [2012] reports an average speed of train and truck (utilization = 80%) between 60 to 70 km/h with time-related costs of € 3.96 per hour (truck) respectively € 3.71 per hour (train) for a full container. The distance-related costs are given with € 4.61 per km respectively € 4.17 per km with handling costs of € 408. The resulting costs for a transport distance of 1,000 km are € 0.29 per tkm respectively € 0.26 per tkm. The resulting costs for a transport distance of 1,000 km based on the report of Planco [2007] are € 0.088 per tkm (truck), € 0.074 per tkm (rail) and € 0.033 per tkm (vessel). The EU-cofunded project COMPASS (2010) reports average costs for trucks, with € 0.105 per tkm, being rather high compared to rail, with € 0.04 to 0.08 per tkm, or small vessels, with € 0.02 to 0.04 per tkm. All these models show somewhat large deviations that may be explained with regard to considered/neglected cost components, compared transport routes or other factors. Independent of total costs of these models, the principal relations are in line with market shares being in favor of truck transport for short to medium distances not exceeding 400 km. Due to availability as well as pre- and end-haulage costs, train transport is competitive for medium to long distances beyond 200 km with advantages for inland navigation due to higher fixed but lower variable costs for distances exceeding 500 to 800 km [Hoffmann, M. et al. 2014].

6.5 Transport cost modelling

For an optimization in a WAMS or an assessment of competitiveness as well as limitations of transport modes, it is necessary to develop a consistent multi-modal transport cost model. For a specific analysis in the Danube corridor, this transport cost model needs to be calibrated to account for current market conditions. The generalized model includes time- and distance-dependent costs for cargo transport by road, rail and inland waterways as well as one-off costs e.g. for loading, unloading, port fees, insurance or logistics according to Formula (60).

$$C_{trans} = C_{time} + C_{dist} + C_{ind} \tag{60}$$

with C_{trans}= total unit transport costs; C_{time}= time-dependent transport costs; C_{dist}= distance-dependent transport costs; C_{ind}= individual one-off costs;

To account for the real market situation, pre- and end-haulage costs for inland navigation and rail transport have to be included. However, external costs of the different transport modes are not considered in this cost model as they are currently not included in market prices. Based on these components, unit transport costs have to be calculated as a function of transport distance for different utilization scenarios and all modes of transport. Due to significant differences in fuel consumption, unit transport cost functions for inland navigation have to be split into upstream and downstream transport. As a simplification, both transport directions have been calculated for the same level of utilization.

The compiled transport cost model in this study for trucks (40 tons) calculates 70 km/h for velocity, time-related costs of € 20 (€ 5 per hour and vehicle + € 15 per hour for driver CEE), fuel consumption from 24 to 34 l/100 km at € 1.4 per litre and tolls of € 0.1 per km. Fixed costs for loading, unloading, waiting time and logistics are assumed with a total of € 5.5 per ton. According to various sources, the load factor of trucks is 60% (pre-/end-haulage) and 80% (line transport). To provide information on different possible scenarios, the transport cost calculation for trucks is based on a load factor of 60, 80 and 100 percent.

The transport costs for rail are based on a full train with 26 wagons, a loading capacity of 837 tons and a total weight of 1,561 tons. Average speed is 40 km/h with a typical load factor of 75%. Furthermore, pre- and end-haulage, each with trucks (60%) in a catchment area of 50 km are assumed as well. Fixed costs for loading/unloading, with € 2.8 per ton, as well as total distance-related costs for train transport are adapted from JANIC [2007] according to the following Formula (61).

$$C_{dist} = 0{,}58 * (weight * distance)^{0{,}74} \tag{61}$$

with C_{dist}= distance-dependent transport costs.

The costs of inland navigation are based on the most common Johann Welker (MGS) vessel type on the upper Danube, with a speed between 7 to 17 km/h depending on river stretch with respect to the direction, up-/downstream, and flow velocity resulting in a difference in energy demand from 300 to 600 kW (based on a typical consumption of 0.24 l/kW with € 1.4 per liter). The vessel speed in backwater sections of hydropower plants, which represent approximately 20 percent of the river, is slightly higher than on free-flowing sections and constitutes the remaining 80 percent of the transport route in the basic model. Time-related costs of vessel and crew are assumed with € 780 per day, with 14 h/day operating time. In addition to travel time, waiting times at locks are assumed with 0.5 h each [Simoner, M. et al 2004; Schwanzer et al. 2010; Bruinsma et al. 2012]. Pre- and end-haulage costs are based on the same assumptions as for rail. Together with at least one day waiting time at each port and one spare day, the transport time is obviously much longer with vessels compared to other modes [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

6.6 Applied transport cost model

Figure 148, Figure 149, Figure 150 and Figure 151 provide an overview of unit transport costs that depend on transport distance and different load factors for different modes of transport. Road and rail transport are compared to the most common single vessel on the upper Danube, i.e. the Johann Welker type (accounting for roughly 50% of transport operations) with deadweight of 1,350 tons. The transport cost model is calibrated mainly for the upper Danube, as consistent data on fleet composition and cost components have been made available. In order to be able to also assess transport processes for the lower Danube, an adaption of the cost model for pushed convoys will be necessary [Hoffmann, M. et al 2014b].

Generally, inland navigation transport downstream is much cheaper, when compared to upstream, due to higher travel speed and lower fuel costs. With a high level of utilization only being possible with bi-directional transport relations, actual market costs will fall between these cost curves. Based on the presented transport cost model, possible savings (budget or transport costs) from improved fairway conditions can also be estimated. Per centimeter of utilized additional vessel draught, the transport capacity increases sufficiently (e.g. 7.8 tons for Johann Welker type vessels) leading to additional revenues and substantial possible savings. For all utilization scenarios, the figures include necessary fairway depth (draught loaded + squat + Underkeel clearance). Thus, for vessel load factors of 40, 50, 60 and 70 percent, fairway depths of 1.78 to 2.18 m, 1.96 to 2.36 m, 2.14 to 2.54 m and 2.32 to 2.72 m would have to be provided by waterway authorities.

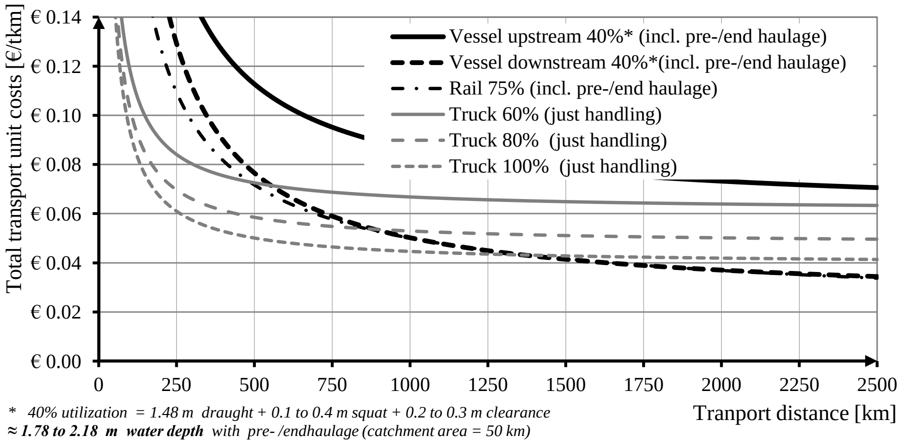
According to the model, direct trains (75%) will be competitive compared to truck transport (80%) for distances beyond 200 to 300 km. If somewhat high pre-/end-haulage (50 km each) costs are considered, the train transport will be competitive only at long distances exceeding 600 to 700 km. The competitiveness of inland navigation is obviously higher downstream and is highly dependent on the load factors realized by the fleet. For typical import, export and transit distances usually exceeding 500 to 800 km, inland navigation will always be cheaper compared to truck transport but is in close competition to rail transport, depending on the individual situation. The results indicate that an average vessel load factor of at least 55 to 60% is necessary in order to stay in the market [Hoffmann, M. et al 2014b].

These results are in line with the responses from navigation companies and further market analysis as well. Thus, inland navigation is currently barely able to stay in the market for average transport distances exceeding 1,000 km, but revenues for necessary reinvestments are critically low. Overageing vessel fleet and equipment are further indicators that confirm these results. Increasing market shares at shorter transport distances call for an average load factor of the vessel fleet exceeding 60% [Hoffmann, M. et al 2014b].

However, such a goal cannot be achieved if actual fairway availability of 2.5 m depth is far below the target conditions of 94% of days/year. For limited maintenance budgets the results clearly confirm that it is far more efficient to concentrate on sufficient fairway depths and minimal widths as compared to a strategy with fixed fairway parameters foreseen by international agreements and recommendations. If external costs are included as well, possible economic savings in inland navigation due to improved fairway conditions will be a few times higher [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014].

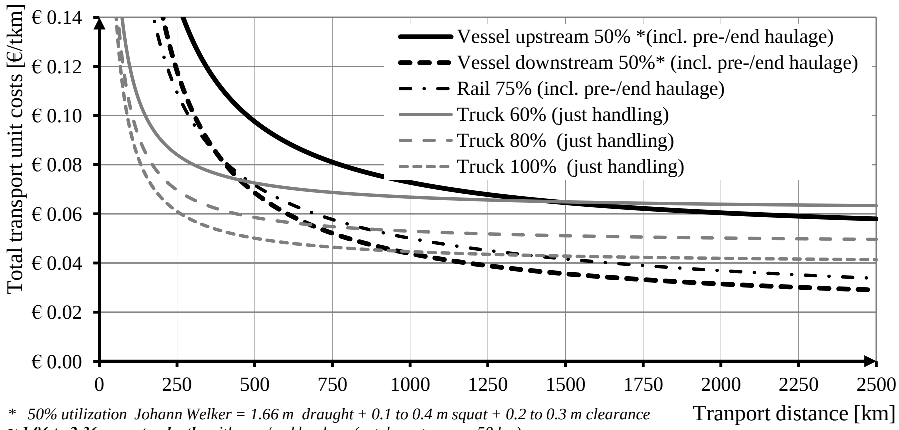
6.7 Transport cost scenarios for different utilization levels

Figure 148: Transport unit costs for Johann Welker vessel type (40% utilization) depending on the transport distance [Hoffmann, M. & Haselbauer, K. & Blab, R. & & Hartl, T. 2014]



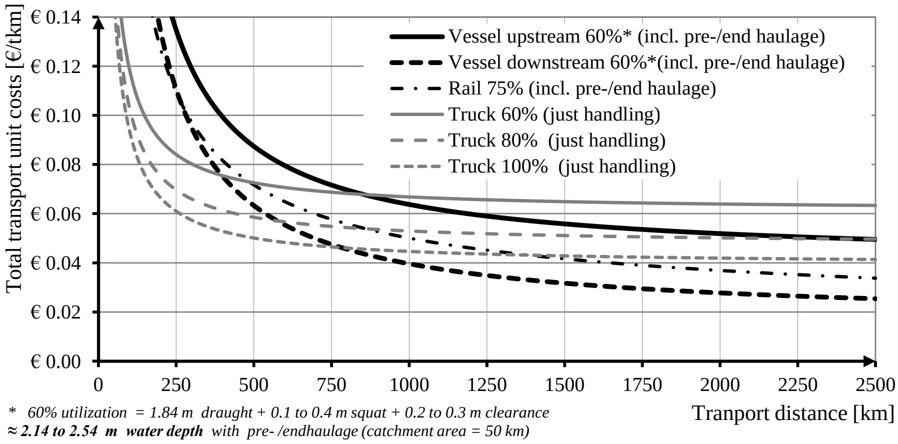
** 40% utilization = 1.48 m draught + 0.1 to 0.4 m squat + 0.2 to 0.3 m clearance*
***≈ 1.78 to 2.18 m water depth** with pre- /endhaulage (catchment area = 50 km)*

Figure 149: Transport unit costs for Johann Welker vessel type (50% utilization) depending on the transport distance [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]



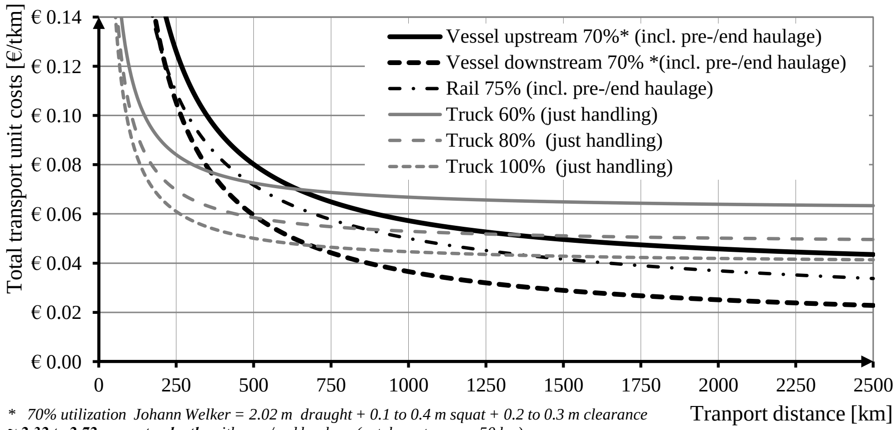
** 50% utilization Johann Welker = 1.66 m draught + 0.1 to 0.4 m squat + 0.2 to 0.3 m clearance*
***≈ 1.96 to 2.36 m water depth** with pre-/end haulage (catchment area = 50 km)*

Figure 150: Transport unit costs for Johann Welker vessel type (60% utilization) depending on the transport distance [Hoffmann, M. & Haselbauer, K. & Blab, R. & & Hartl, T. 2014]



** 60% utilization = 1.84 m draught + 0.1 to 0.4 m squat + 0.2 to 0.3 m clearance*
***≈ 2.14 to 2.54 m water depth** with pre- /endhaulage (catchment area = 50 km)*

Figure 151: Transport unit costs for Johann Welker vessel type (70% utilization) depending on the transport distance [Hoffmann, M. & Haselbauer, K. & Blab, R. & Hartl, T. 2014]



** 70% utilization Johann Welker = 2.02 m draught + 0.1 to 0.4 m squat + 0.2 to 0.3 m clearance*
***≈ 2.32 to 2.72 m water depth** with pre-/end haulage (catchment area = 50 km)*

6.8 Optimization options

In general, different optimization objectives can be pursued with the presented WAMS. This approach is based on the comparison of a given availability target and the actual availability performance of a river stretch in days per year. If this performance is insufficient, fairway availability has to be improved by implementing maintenance measures which may be described by a measure cost surface if all possible combinations of fairway width and depth are considered. With increasing target fairway parameters, the required measure extent and costs are increasing, resulting in a rising concave maintenance measure cost surface. The resulting availability of fairway width and depth on a transport route finally affects transport costs. This total annual transport cost, for a transport route with a given combination of fairway width and depth, are a result of summing up utilization-related transport costs throughout a year. If these annual transport costs are calculated for any combination of fairway width and depth, the resulting concave transport cost surface will decrease with increasing fairway dimensions (Figure 152).

Starting with given fairway dimensions (e.g. DC 2013, UNECE 1996) as input parameter and resulting measure costs, annual budget requirements for waterway authorities can be calculated. With availability-based transport costs and fleet composition at hand, the resulting transport costs for any level of availability may be calculated. A comparison with actual available budget indicates which maintenance target (LOS) can be achieved. The approach provides the means to calculate transport cost savings for any investment strategy and budget.

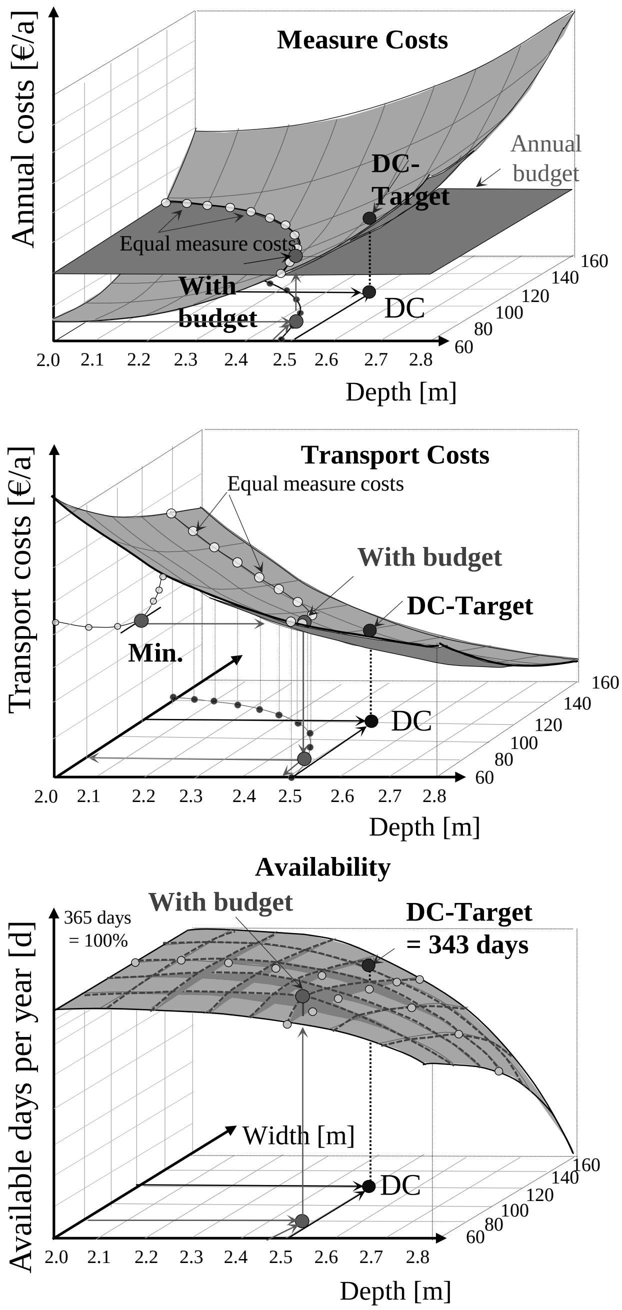


Figure 152: Schematic illustration of optimization based on availability, annual measure costs and resulting annual transport costs.

In general, measures may be optimized regarding different fairway parameters, leading to specific combinations of continuous fairway widths and depths within the same budget. The resulting combinations may also be modeled as an intersection of the horizontal annual budget surface with the increasing measure cost surface for achieving increasing availability of fairway depths and widths. If this intersection line is projected on the resulting transport costs surface, then the optimal combination of fairway width and depth with minimal transport costs within the given budget can be found (Figure 152). Logically, recommended fairway parameters (e.g. DC 2013) that appear above the availability surface for any given budget cannot be achieved without further funding [Haselbauer, K. et al. 2014].

6.9 Optimized fairway parameters

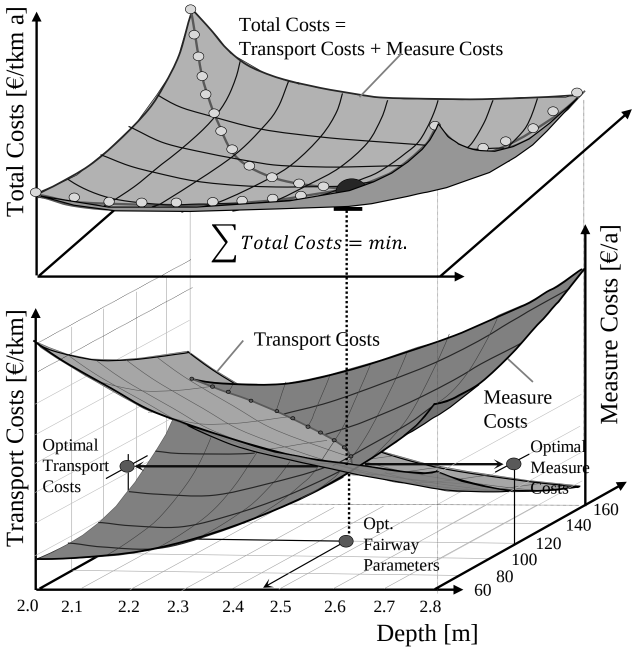


Figure 153: Schematic illustration of optimizing target fairway parameters based on minimal total annual costs of measures and transport

For infrastructure operators acting as private-sector enterprises, the main focus will mainly be on minimizing expenditures for operation, maintenance and engineering measures. In a competitive market, on the other hand, it is necessary for individual players to increase market shares and not to lose market shares to other modes of transport. In order to find favorable fairway conditions for waterway operators and the navigation and shipping industry, annual maintenance and transport costs, for each combination of fairway depth and width, have to be calculated. For an overall optimum of both measure and transport costs, the annual measure costs and availability-based annual transport costs for the same fairway width and depth have to be added (Figure 153). At the point of the overall minimum of total measure and transport costs, the optimal combination of fairway parameters is found for the given situation according to Formula (62) and (63).

$$C_{total}(w,d) = C_m(w,d) + C_{tr}(w,d) \tag{62}$$

$$\sum_{i=0}^{n} C_{total}(w_i, d_i) = \min! \tag{63}$$

for each combination of width w_i and depth d_i in meters; with C_{total} = total annual costs, C_m= annual measure costs, C_{tr}= annual transport cost and $0<d_i< 4$ m , $0<w_i<250$ m

In addition to optimal fairway parameters, resulting measure costs and an estimation of transport costs on the market may be identified as well. If measure costs are unknown it would still be possible to describe the impact on transport costs based on any given availability target. The same would hold true for measure costs if transports costs are unknown. Modelling of actual transport costs is a difficult task due to a number of factors affecting the results in the transport market. Nevertheless, even a rough estimation of transport costs can provide useful results if measures are only compared to each other. With a more accurate cost model, the impact of measures on the transport market may be assessed as well. The accumulation of all costs of optimal measures on all critical sections for achieving predefined continuous fairway conditions leads to a certain necessary budget for achieving a certain set of conditions. With an infinite number of possible combinations of fairway depth and width in days per year that might be achieved within different budgets, it is very unlikely that any set of predefined or recommended fairway parameters would be optimal. Even if just measure costs and resulting transport costs are considered, it would be rather pure chance that recommended fairway parameters are the same as optimal fairway parameters at the point with the lowest measure and transport costs combined. Further improvements of the optimization approach could also include an environmental assessment of planned measures. This would allow a balanced view between local impacts of measures (e.g. on habitats) and general impacts on the environment within the transport corridor (increasing CO_2 emissions due to shifting effects to modes of transport [Haselbauer, K. et al. 2014].

7 SUMMARY AND CONCLUSIONS

In Europe, the modern transportation infrastructures of road and rail are well developed, with the majority of the high-speed networks having been constructed decades ago. In contrast, inland waterways have played an important role for centuries, especially in goods transport. The preservation and development of these infrastructures are essential for the European economic system and the welfare of Europe in general. With increasing age of infrastructure systems, traffic loads, climate, and environmental conditions are leading to ageing and structural deterioration. If appropriate measures are not implemented in time, these processes may result in system failure with severe consequences for infrastructure operators and customers as a worst-case scenario. Infrastructure asset management systems provide the required systematic approach covering all main tasks within their responsibility from “cradle to grave”, based on comprehensive life cycle approaches. Such up-to-date approaches have already been implemented for road and rail infrastructures, to some extent, but are still missing on inland waterways. Contradictory to desired developments, with an increasing share of inland navigation on the transport market, current development trends show gradually declining importance compared to road and rail.

As inland waterways are a linear mode of transport that allow for no detours or alternative routes except for another mode of transport or unloading parts of the cargo to another vessel, a single shallow section with low fairway depths limits utilization and efficiency of goods transport. For this transport system with a serial arrangement of shallow sections, the section with the lowest fairway depth is decisive for the utilization of the majority of the fleet. If the availability of certain fairway parameters cannot be guaranteed on an entire transport route throughout the year, it is considered unreliable compared to other modes despite comparably low transport costs. The comparison of the recommendations from the Danube Commission with stated fairway availability from waterway agencies revealed large disparities between riparian countries, and are clearly below agreed standards in general. However, it is quite clear that positive impacts of possible investments in fairway availability will be very limited if the necessary riverbed surveying and operation activities cannot be performed due to restricted budgets in other riparian countries, outdated equipment or a lack of staff. As a bottom line, even the best riverbed surveying and fairway marking activities cannot change the physical fact that one single critical (i.e. shallow or narrow) section cannot be passed with a competitive utilization of loading capacity.

In order to obtain uniform condition parameters (e.g. fairway depth) for an entire transnational waterway, developing a comprehensive asset management approach that accounts for all essential characteristics, followed by consequent implementation, is mandatory. Such an asset management approach, capable of providing all necessary tasks, has been developed at the Vienna University of Technology – Institute of Transportation on behalf of the Austrian waterway authority VIADONAU within a pilot project. With infrastructure quality determined by availability and reliability of fairway parameters as the center piece of the presented approach, it is possible to determine the impact and efficiency of each possible measure. Thus, this new approach allows for one to account for all aspects of development, maintenance, rehabilitation and replacement of waterway assets based on a comprehensive life cycle costing approach. This pilot project also included the development of a software tool including the core tasks (e.g. survey processing, availability analysis and optimization) of dredging measures. Based on an extension and application of common methods of existing asset management approaches, this new approach is unique for inland waterways.

Based on periodic riverbed surveys, current water levels and amount of discharge, the impact of maintenance and river engineering works on the availability of fairway widths and depths can be modeled, allowing for a calculation of real time availability from cross-sectional profiles up to entire transport routes. An innovative alert system based on an empirically derived backfilling behavior of critical bottlenecks (e.g. after implementation of dredging measures) allows for a determination of impact duration, optimization of timing and thus an efficient measure implementation. To highlight the importance of continuous uniform fairway depth, several examples of draught loads for typical vessels on the Danube are provided. As an example, the most common vessel type, “Johann Welker”, has a maximum loading capacity of roughly 1400 t corresponding to a draught of 2.6 m. One additional centimeter of fairway depth equals 7 tons of additional goods and has, therefore, a substantial impact on the utilization of the Danube vessel fleet if there are several weeks a year with possible utilization falling below 50%, in order to pass certain shallow sections. Fairway width would also be an important issue if the number of vessels would be a few times higher, as this would lead to waiting times in narrow sections. However, at the current level of utilization of the Danube, the frequency of encountering other vessels is rather low and thus possible waiting times, and their impact on costs, are rather negligible. In summary, a guaranteed minimum availability of fairway widths and depths is crucial both for tendering of transport contracts and planning of individual transport trips, in order to stay competitive compared to other modes of transport.

The findings of the research indicate that maintaining a navigational channel on a recommended maximum width (compare recommendations DC 2013) is to be considered extremely costly, since evaluations of dredging volumes have highlighted a substantial progressive increase in dredging costs with increasing fairway width. Even in cases with current low encounter probability of pushed convoys with four barges (max. convoy size on the upper Danube) in very few narrow sections, the resulting waiting times for users are marginal compared to overall transport time. Thus, avoiding a few minutes, or even one to two hours’ time loss as a worst case scenario, on trips with an average transport duration of one to three weeks would come with comparably very high costs for waterway operators.

However, even if one national waterway operator that does not implement necessary measures, and thus fail to provide certain minimum availability conditions and reliable information, this will limit the efficiency of all other investments on the entire Danube. Thus, the results of the research indicate that only concerted actions on a transnational level of all waterway authorities and stakeholders under a common strategy will lead to efficient investments. The implementation of such a harmonized common strategy, together with an implementation of necessary maintenance and river engineering works, will be crucial for inland navigation and can be described as the overarching goal for the future of the Danube River as a competitive mode of transport in the heart of Europe. In summary, positive developments in waterway transport are unlikely, despite considerable efforts in the majority of riparian countries, if necessary maintenance measures are not conducted in certain other countries. The presented methodological framework and developed waterway asset management system WAMS takes the first steps towards this goal, and was already successfully implemented on the Austrian stretch of the Danube. The empirical data suggests that the developed approach is sound and an implementation of the developed waterway asset management system on the entire Danube River is feasible.

8 OUTLOOK

Waterway asset management as a research field comprises an extremely broad subject. The development of the presented approach started in autumn 2012. As an initial result, a number of methodological foundations have been described. These foundations are already implemented in a software tool and have been verified based on the pilot application on the 350 km Danube River stretch in Austria. The next steps in development will include further analytical capabilities and management functionalities in the WAMS - Software. As an example, sedimentation management, river engineering woks and traffic analysis functionalities will be added in the next step. Furthermore, implementing a more dynamic approach towards signalization and marking activities will lower the time needed between fairway information updates and increase possible utilization. Further possible features of the WAMS - Software include prediction models of the fairway condition based on empiric performance functions of water levels and riverbed development. Also, an in-depth analysis of transport processes (transport costs and fleet model) and an integrated route planner for inland navigation (based on comprehensive depth information of entire transport routes) would be valuable additions. Finally, including a systematic assessment of environmental impact of all relevant measures in the optimization process will be a challenging, but important task for the future.

9 APPENDIX

9.1 List of figures

9.2 List of tables

9.3 References

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