HYDROLOGY AND HYDRAULIC ANALYSIS FOR THE DIVERSION OF THE COCOLI RIVER

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ABSTRACT:

The excavation and construction of the new locks on the Pacific side for the expansion of the Panama Canal, required before its start, a series of auxiliary works to avoid affecting its development.

One of the immediate problems that was detected and that could negatively affect the development of the expansion works was the course of the Cocoli River that discharged its waters into the Miraflores Lake and that would be intercepted in its course by the projected alignment and construction of the new locks.

The solution to this problem, it was to change the trajectory of the waters of the Cocolí River through the design and construction of a diversion channel, which was part of the Canal Expansion Project (PAC-2). This prevented that the future excavations from coming into conflicts with the schedule of activities that would take place in the area.

The design of the channel of diversion of the Cocolí River considered two stages: the development of the hydrological study and the integral hydraulic analysis of the projected channel and its impact along its trajectory.

In the hydrological study, the design flow rates of the Cocolí River were estimated up to the diversion site and three tributaries that run naturally through the area and that will be affected in their natural course by the projected alignment of the diversion channel. In order to estimate the design flows, preliminary studies were used, but due to the magnitude of the project, they were expanded using hydrological modeling techniques, which were complemented with field inspections.

The hydraulic analysis of the diversion channel contemplated the integral study of all the existing components and structures and those that were design, the alignment of the new channel and the problems that could introduced in the area, due to the build of the new channel, the flood plains and determination of the water profiles along diversion channel, in order to model the whole system to determine all problems and recommend their solutions in the most economical way.

Within the hydraulic design of the diversion channel, it was contemplated to take advantage of part of the existing channels to reduce the work of excavation. Likewise, the construction of a vehicular bridge was analyzed to give access to the west side of the Panama Canal and to the contractors who developed the excavation and construction works of the new locks.

Keyword: River, diversion channel, Hydrology and Hydraulic Modelling, design.

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Topics: 1.1 - Waterway infrastructures: locks, weirs, river banks

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1. BACKGROUND:

The Cocolí River Diversion Channel (DCP) Project is proposed to be developed in an area that currently has no permanent human settlements. During the period of permanence of the United States in Panama, it was a military reserve and towards the Nuevo Emperador there was an area of military maneuvers where shooting practices were carried out. After the Canal was transferred to the Panamanian Administration, part of the Cocolí area became the Canal's operating area and was established as Type I Operating Area and Type II Operating Area. The diversion channel will cross these areas, but it will also cross other areas that are not for the exclusive use of the ACP.

The area that will cross the diversion channel, the only economic activity that takes place is associated with the operation of the Panama Canal, mainly the disposal of dredging materials in two deposits that are crossed by the Velázquez River.

2. LOCALIZATION AND GENERAL DESCRIPTION OF THE COCOLÍ RIVER DIVERSION CHANNEL PROJECT:

The project of the diversion channel of the Cocolí River, is located in the Corregimiento de Arraiján, District of Arraiján, province of Panama, at the entrance of the Pacific Ocean of the Panama Canal, on the west side, between coordinates 991 800 and 993 050 of North latitude and 652 750 and 655 900 of western longitude.

The population centers closest to the project are the District of Arraiján located to the west and the city of Panama to the southeast. The center of the city of Panama is approximately 7 km in a straight line northeast of the project. In front of the confluence of the Cocolí Canal with the Velázquez River, a high-value residential complex is currently being developed, which includes its own Golf Club, known as El Tucán Country Club.

The project is located within basin 142, between the Caimito and Juan Diaz rivers and has a drainage area of 29.5 km2, up to the diversion site.

Figure No. 1 shows the general location of the diversion channel of the Cocolí River.

3. SCHEME OF THE PROJECT

In Figure No. 2, the project outline of the Cocolí River diversion channel is presented and its main components are described below.

The scheme of the Diversion Channel Project (DCP) of the Cocolí River, consists since its inception in Lake Cocolí of the following components.

- A diversion levee dam at least 17.00 meters high with the base located on the fluvial channel of the Cocolí River. Figure 3 shows a picture of Lake Cocolí with the beginning of the diversion channel.
- Diversion channel starting at Lake Cocolí at an elevation of 10.87 m PLD with a length of approximately 3.5 km, trapezoidal section with slopes of 1: 2 and variable hearth width, starting at 15 meters and ending at 30.00 m in the Pacific Ocean.
- The trajectory of the channel will have to cross the existing road that leads to the K9 polygon and the Brujas roadway, where due to the construction of the channel, a concrete drainage system of 3.00 mx 3.00 m should be demolished, to be replaced by a bridge. Figure No 4 shows a photo of the drainage system that will be replaced by a new bridge.
- From station 1 K + 080.00 to 1 K + 440.00, it is necessary to build approximately 360 meters of levees with an average height of 1.80 m on both sides, in order to contain the waters inside the channel, from the station 2 K +60.00 to 2 K + 100.00 (40.00 m in length) and from 2 K + 460.00 to 2 K + 580.00 (120.00 m in length).
- From station 2 K + 040.00, because the channel crosses and affects the flow of three existing streams that are the Victoria Sur, Victoria Norte and the Velázquez River, the DCP must be supplemented with accessory works that includes channeling and coating from the entrances of the tributaries to the diversion channel. Figure 5, 6 and 7 show the photos of the Victoria Norte, Victoria Sur and Velázquez rivers.
- At the confluence of the Velázquez River with the diversion channel, a buffer pond is contemplated to reduce turbulence.

Two critical zones have been identified, which are the area of the levees and the confluence of the Cocolí DCP and the Velázquez River. In this zone, depending on the use given to adjacent lands, dykes can be built or not.

In the recommendations and solutions, a detailed description of the work that must be done along the Cocolí river diversion channel is presented.

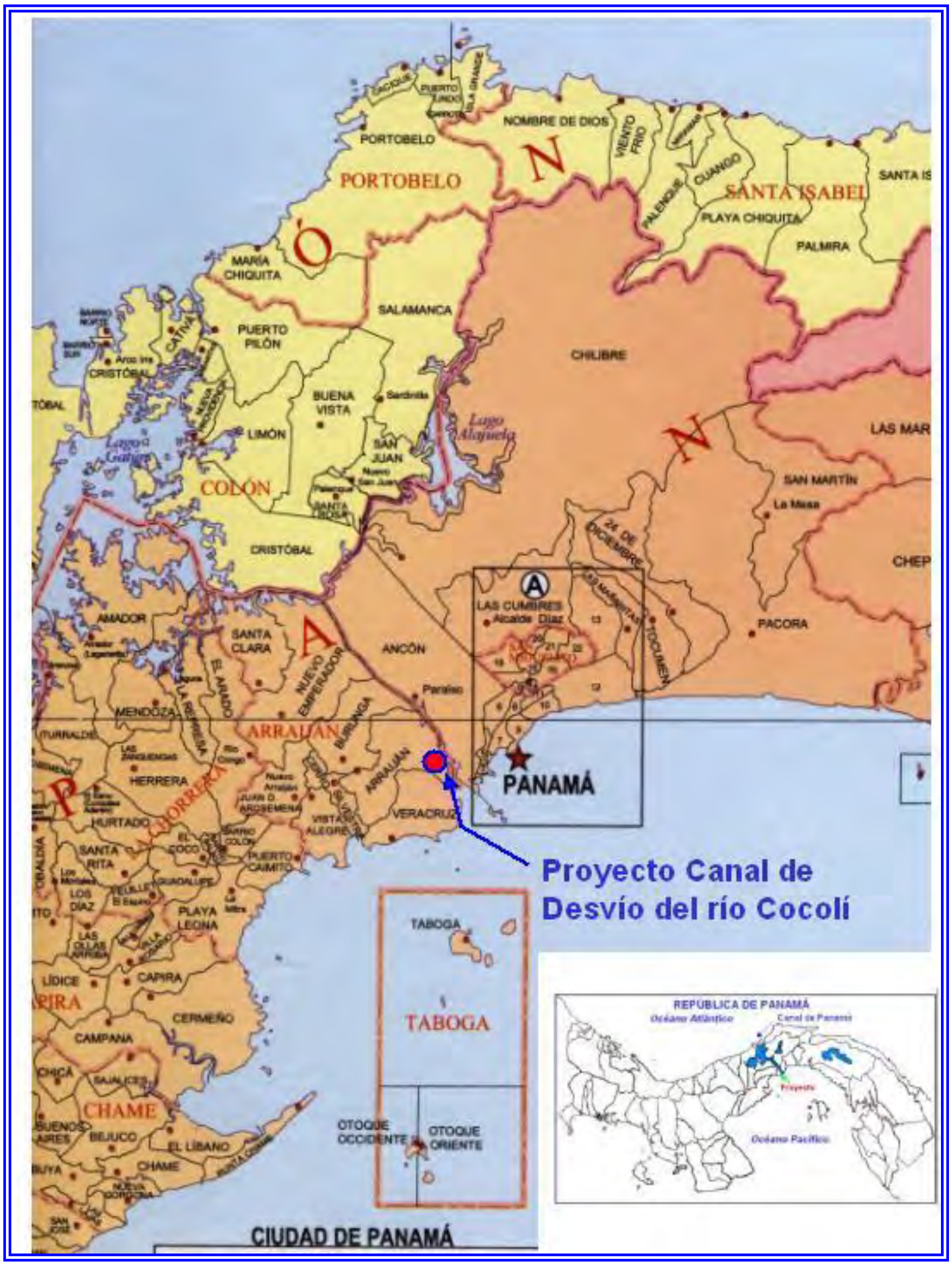

Figure 1. General location of the diversion channel of the Cocolí River.

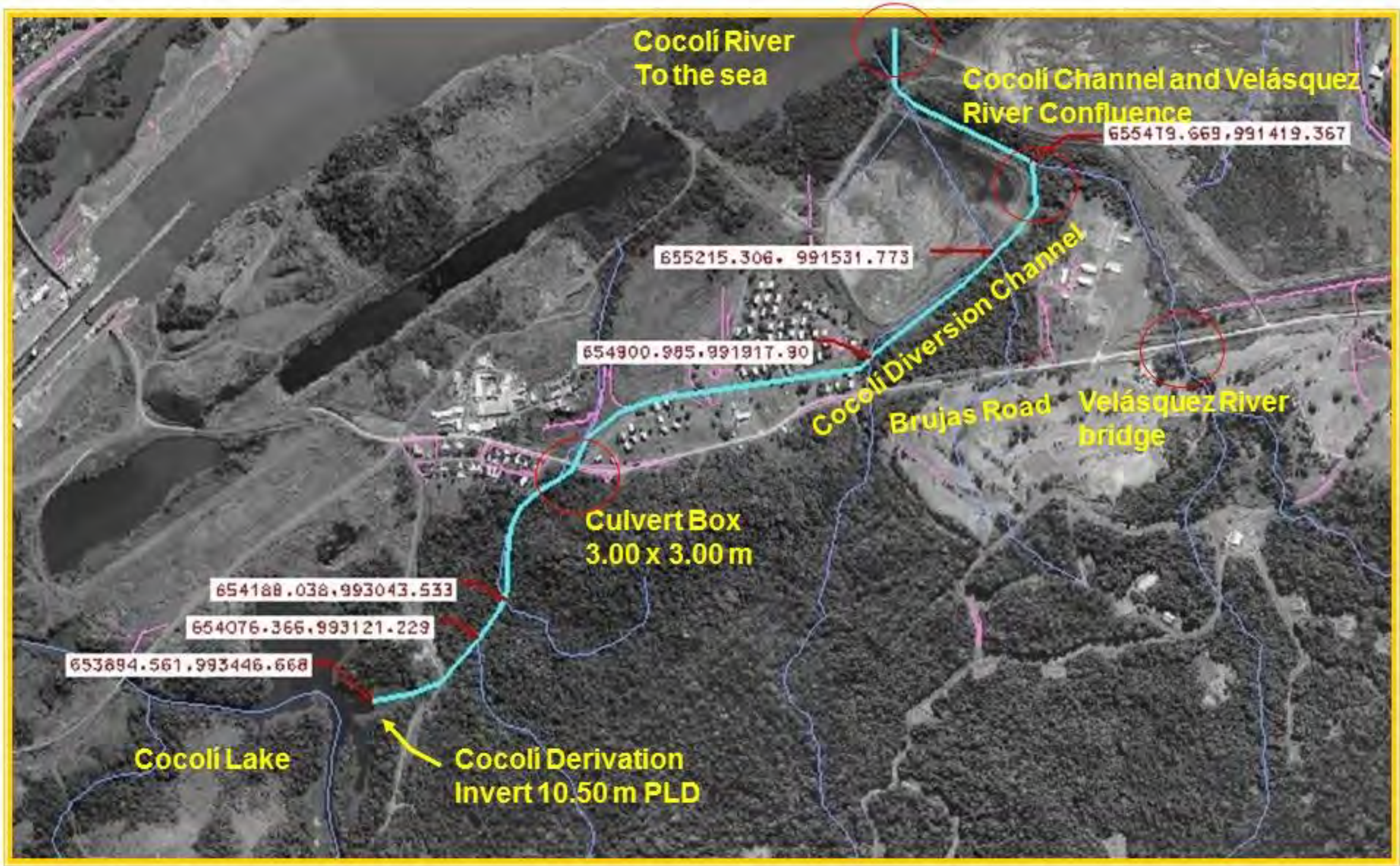

Figure 2. Scheme of the diversion channel of the Cocolí River.

Figure 3. Confluence of the Cocolí river with the Miraflores lake and outline of the future entrance to the diversion channel. LPalma / June 2007

Figure 4. Culvert concrete drainage system 3.00 x 3.00 on the road to Brujas, which will have to be removed to run the diversion channel of the Cocolí river. JACM / June 2007

Figure 5. Horseshoe culvert in the Victoria Norte creek the Brujas road. View from upstream in the channel of the gulch. Dimensions 3.00 x 3.00 m. JACM / June 2007

Figure 6. Horseshoe culvert in the Victoria Sur gorge, on the Brujas road. View from downstream in the channel of the gulch. Dimensions 1.80 x 1.80 m. JACM.

Figure 7. View of the bridge over the Velázquez River, on the Brujas road looking towards Cocolí. JACM.

4. GENERAL DESCRIPTION OF THE WATERSHED 142 THAT COCOLÍ RIVER SUBBASIN IS PART

According to the Watershed distributions of the Republic of Panama, the sub-basin of the Cocolí River is located within Watershed 142, rivers between the basins of the Caimito River and Juan Díaz River, on the Pacific side of the country, on the west bank of the Panama Canal and its waters pour into Lake Miraflores.

The sub-basin has a circular shape and its topography is quite homogeneous because there are no significant elevation changes from the headwaters of the Cocolí River to its mouth.

The landscape of the sub-basin is dominated in its upper part to the northwest by the highest point which is an unnamed hill with an elevation of 257 meters above sea level. The Cocolí River from its source has an approximate elevation of 220 meters above sea level.

The Cocolí River Diversion Channel Project starts at the end of the Cocolí sub-basin, in the lower part with elevations between 15.00 and 0.00 m PLD.

5. BASIC INFORMATION

The basic information for the development of the hydrological and hydraulic study was obtained from three main sources:

- Meteorological and hydrological information
- Existing cartographic information
- Topographic surveys

6. RAINFALL REGIME OF THE COCOLÍ RIVER SUBBASIN

The rainfall regime of the Cocolí sub-basin is influenced by the rainy regime of the Pacific side, which is characterized by two well-defined seasons. The dry season that usually goes from mid-December to April and the rainy season from mid-April to December.

Within the rainy season there is a decrease in rainfall between the month of July and August, which is caused by the annual movement of the Inter Tropical Convergence Zone (ITCZ), when it is farthest from the Isthmus of Panama, a phenomenon known as the Veranillo de San Juan or Canícula. The ITCZ is the confluence zone of the trade winds of both hemispheres, North and South. It is an area of light and variable winds, unstable air and strong convective developments, with intense rains. When the Intertropical Convergence Zone moves from North to South, rainfall increases again, with October being the rainiest.

The seasonal distribution of rainfall in the Cocolí sub-basin is controlled by the ITCZ, however, the totals that occur in any point of the country depend on factors such as elevation, relief, distance to the mountain range, exposure to prevailing winds, etc.

In Figures 8 and 9, the rainfall distribution is presented for the rain stations closest to the project, which are Balboa and Pedro Miguel.

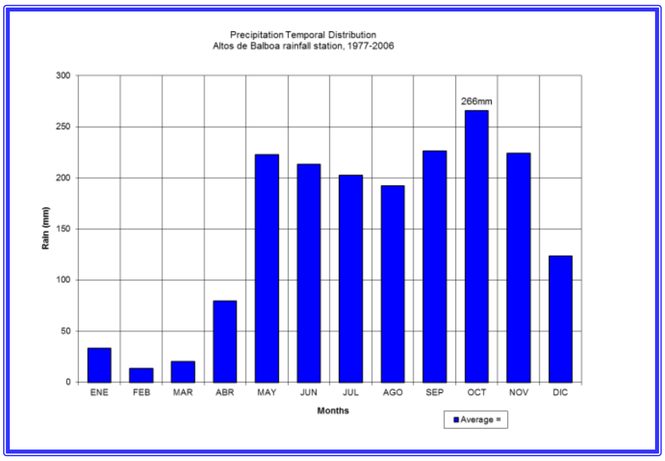

Figure 8. Temporal distribution of rainfall in the Altos de Balboa rainfall station, period 1977-2006.

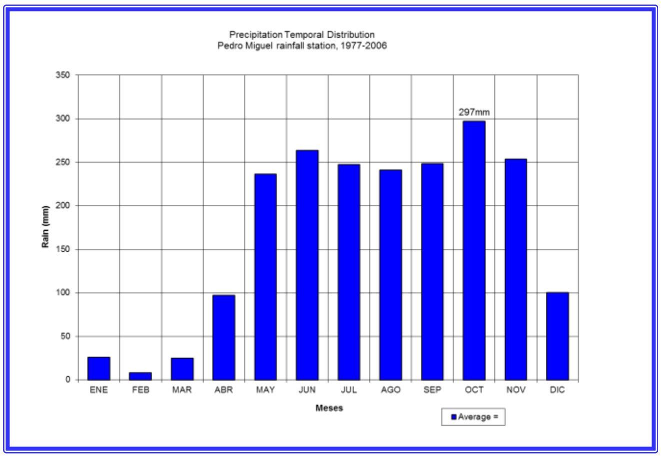

Figure 9. Temporal distribution of rainfall in Pedro Miguel rainfall station, period 1977-2006.

7. HYDROLOGICAL ANALYSIS

The purpose of the hydrological study was to examine all the components of the surface hydrology of the hydrographic basin that feeds the Cocolí River, with the purpose of estimating primarily their design flows, in addition to analyzing other aspects such as the distribution of monthly average flows.

To complement the analysis, an inspection and field survey was carried out to sites of the Cocolí river basin previously selected, which were selected as preliminary reconnaissance of the topographic mosaics available at a scale of 1: 50000 and 1: 25000. The objective of this inspection and survey was to compare the results obtained through hydrological modeling, with the calculations made as a result of the field surveys.

7.1. Design Flows:

As there are no references of measurements or systematic readings in the Cocolí River, we proceeded to estimate the design flows by indirect methods.

For the initial design of the Cocolí DCP, the Geotechnical Section preliminarily used the maximum instantaneous flows that appear in the report "ESTIMATION OF THE MAXIMUM FLOWS OF THE COCOLÍ RIVER", analysis carried out in July 2003 by the Hydrologist Tamara Muñoz, but due To the extent and scope of the Cocolí river diversion channel project, the study was updated and expanded. The tool used for the extension of the study was the HEC-HMS hydrological program developed by the Engineering Corps of the Hydrological Engineering Center of the United States, which is free of charge.

The flows were estimated for return periods of 5, 20, and 50 and 100 years, for the Cocolí River and the Victoria Sur, Victoria Norte, Victoria 3, Victoria 4 and the Velázquez rivers, which will be affected by the deviation channel project when it has been built.

7.2. Description of the HEC-HMS Model

The HEC-HMS system was designed to simulate the precipitation runoff processes of a basin and was developed by the Corps of Engineers in the early 60's and its predecessor is known as HEC-1. The HEC-1 or HEC-HMS, is one of the most popular events simulation programs, which can be used for free. The version used for the analysis of the design flows is 3.1.0.

The acronym HEC stands for Hydrologic Engineering Center, which is the research center of the United States military engineering corps, located in Davis, California.

7.3. Requirements of the HEC-HMS Model

The requirements of the model depend on what is required to model and on the hydrometeorological information available to feed the model.

For this particular case, what is required to be developed is the design hydrograph of the project, which is why the application of the HEC-HMS model was considered appropriate, since it is an event simulation model. An event model has the capacity to reproduce a single storm, given certain physical parameters of the basin under analysis.

As there are no hydrometric records in the Cocolí basin, we proceeded to estimate the flows using the "Alternate Block Method" and the flood transit for each Subbasin using kinematic wave.

The model requirements for this method are basically physical characteristics such as, drainage surface and time of concentration of the basin and its sub-basins, length and slope of the channel and slope of the landscape and estimation of number of curve.

All this information is determined by means of topographic mosaics, aerial photographs and field inspections. Once these data were obtained, we proceeded to estimate the time of concentration and the design rain for the basin for different return periods.

One of the limitations of this method is that you do not have a station or control point at the exit of the basin to calibrate the model. For this reason, an estimate was made of a flow of the Cocolí River using the traces left by a recent flood (May 2007) and an arm of the Cocolí River, proceeding in both cases to estimate its flow using the area-slope method.

7.4. Methodology:

The HEC-HMS model is designed to simulate the surface runoff that results from a rain, through the representation of a basin as a system of interconnected components. Each component can individually simulate an aspect of the rainwater runoff process within an area or sub-basin; the components include the surface runoff of the subarea, the channels and the reservoirs; each component is represented by a set of parameters that specifies the particular characteristics of the component and the mathematical relationships that describe its physical processes. The final results of the modeling process are the output hydrographs or direct surface runoff for each previously specified subarea. Figure 10 shows the HEC-HMS scheme for the Cocolí river basin. The basin was subdivided into 9 small sub-basins.

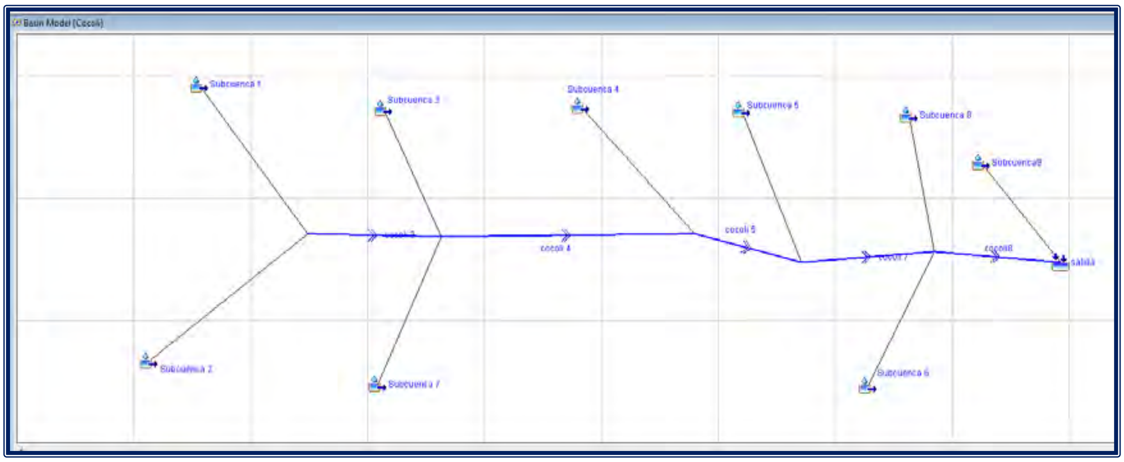

Figure 10. HEC-HMS scheme for the Cocolí river basin.

The surface runoff component for a sub-area is used to present the movement of water over the surface of the land to the riverbeds and streams. The input of this component is a precipitation histogram, which was designed by the alternate block method. The excess of rain is calculated by subtracting the infiltration and losses by arrest, and in our case the Soil Conservation Services (SCS) curve number method was selected and alternatively the kinematic wave model was used to calculate the runoff hydrographs in the sub-basins.

The flood routing component represents the movement of flood waves in the channels. The input of this component is the hydrograph obtained upstream that resulted from the individual or combined combinations of the runoff of the subareas, the transit of flows or the derivations. For the flood routing of the Cocolí, the kinematic wave method was used.

Alternate block method

The alternate block method is a simple way to develop the design histogram using the Intensity-Duration-Frequency (IDF) curves of the station closest to the project under study. The design histogram generated by this method determines the depth of precipitation that occurs in n intervals of successive times of duration Δt over a duration of $Td = n\Delta t$. After selecting the design return period and the time interval □t, the intensity is read from the IDF curve or otherwise the equation generated for the curve is applied, for each of the durations for each Δt, 2 Δt, $3\Delta t$ ..., and the corresponding depth of precipitation is found by multiplying intensity and duration. Determining the difference between the successive values of depth of rain, is the total amount of precipitation that must be added for each unit of time Δt. These increments or blocks are rearranged in a temporal sequence so that the maximum intensity occurs in the center of the required duration *Td* and the blocks are in descending order alternately to the right and to the left of the central block in order to form the histogram of project design.

We analyzed the IDF curves of the closest rainy seasons, which are considered the most influential in the Cocolí river basin, which are Balboa Heights and Pedro Miguel. The Balboa curves developed for the Ministry of Public Works (MOP) by Federico Guardia and Consultores in 1972 were used as a reference, for a period of 57 years and those of the study "Analysis of Intensity Duration and Frequencies, Maximum Rain Events Annual (1972-1999), Panama Canal Basin - Eastern Region ", developed by Maritza Chandeck Monteza in 2001. Both IDF curves were compared, and it was observed that the ACP curves compared to those of the MOP, they present differences. The main difference is that for the same time interval, the rain intensity is lower in the curves developed by the ACP for the intervals between 1 to 60 minutes. For the 60 to 120 minute intervals, the rainfall intensity of the ACP curves is slightly higher.

One of the theses that is handled is that from rainfall records until 1972, rainfall intensities could be obtained at 5 minute intervals and from 1972 onwards, the minimum registration interval is 15 minutes. At a shorter time interval, rainfall intensities are greater and decreasing with respect to time.

For the analysis of the Cocolí DCP, the IDF curves recommended by the MOP were used, considering that the values of these curves are more critical for the channel design.

In Table 1, the values of the IDF curves of the Balboa station are presented and in Figure No. 11 the Curve Intensity-Duration-Frequency (IDF) is presented.

Table 2 shows the maximum rainfall data obtained through the alternate blocks to develop the design storm's histogram, which was used to estimate the design flood for a return period of 100 years and Figure 12, the graph of the design histogram for a 100-year storm is presented.

Table 1. Summary of intensities to determine the IDF curves for Altos de Balboa rainfall station, period 1921-1986 (57 years).

Tc (min)	Return Periods T (years)							
	2	5	10	20	25	30	50	100*
5	6.68	7.17	7.88	8.50	8.81	9.02	9.74	11.21
10	5.82	6.39	7.02	7.60	7.87	8.04	8.60	9.74
15	5.16	5.76	6.33	6.87	7.12	7.25	7.71	8.60
30	3.85	4.45	4.89	5.33	5.52	5.61	5.87	6.38
45	3.07	3.63	3.99	4.35	4.51	4.57	4.74	5.07
60	2.55	3.06	3.36	3.68	3.81	3.85	3.98	4.20
120	1.52	1.88	2.07	2.27	2.36	2.37	2.42	2.50

* For the return period of 100 years a formula has been estimated depending on the curves of the other periods of return and their behavior

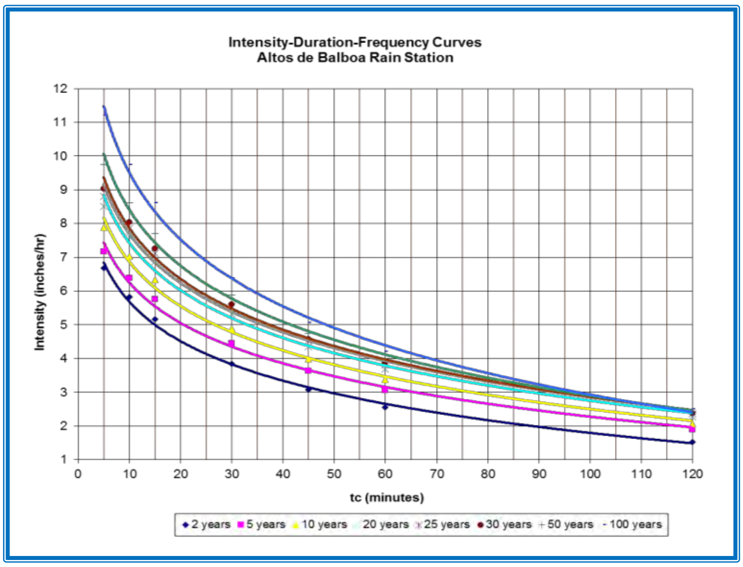

Figure 11. Intensity Duration Frequency (IDF) curve for Altos de Balboa station.

Table 2. Histogram of the rain developed in 10-minute increments for a 100-year storm and 120-minute duration for the Cocoli river basin using the alternate block method, Altos de Balboa station.

Tc (min)	T (hr)	Intensity Tr = 100 y	Inten. (mm/h)	Acum. rain (mm)	Rain (mm)	Histogram (mm)
10	0.17	9.74	247.32	41.22	41.22	2.48
20	0.33	7.71	195.79	65.26	24.04	3.44
30	0.50	6.38	162.03	81.02	15.75	5.09
40	0.67	5.44	138.21	92.14	11.12	8.27
50	0.83	4.74	120.49	100.41	8.27	15.75
60	1.00	4.20	106.80	106.80	6.39	41.22
70	1.17	3.78	95.90	111.88	5.09	24.04
80	1.33	3.43	87.02	116.02	4.14	11.12
90	1.50	3.14	79.64	119.47	3.44	6.39
100	1.67	2.89	73.42	122.37	2.90	4.14
110	1.83	2.68	68.10	124.85	2.48	2.90
120	2.00	2.50	63.50	127.00	2.15	2.15

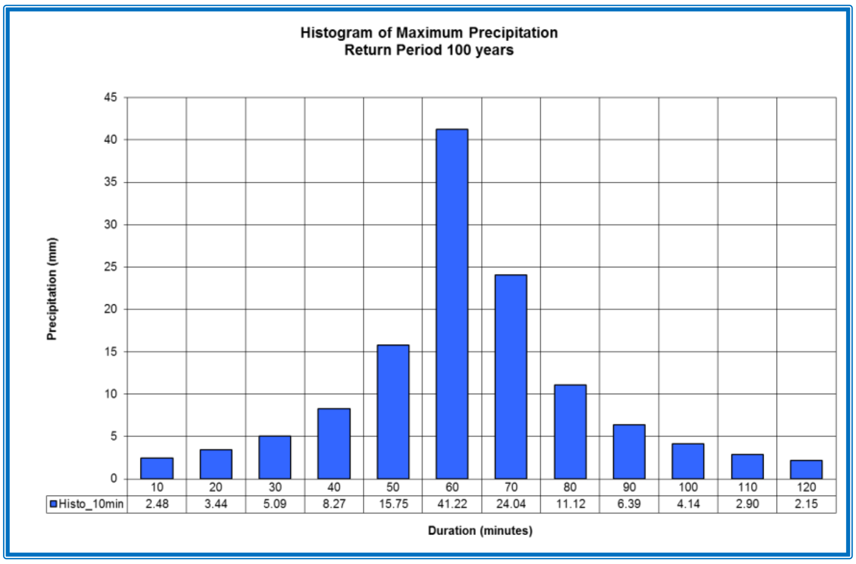

Figure 12. Histogram of maximum precipitation for a return period of 100 years determined using the alternative blocks method.

7.5. Run of HEC-HMS Model:

Once all the physical parameters and rainfall histograms required by the HEC-HMS model were obtained, the model was assembled. The Cocolí river basin was subdivided into 9 sub-basins (Figure 10) and 4 stretches of the river were defined to make the routing of the floods. The layout and measurement of all sub-basins of the project was obtained by measuring the mosaics 1: 50000 of the National Geographical Institute Tommy Guardia. From the mosaics, the physical characteristics of the basin were also obtained as the drainage area, the length of the channels, the average slope of the channels and the slopes perpendicular to the river bed for each sub-basin.

With these parameters we proceeded to determine the time of concentration (Tc) for the entire basin and for each sub-basin, adopting the Kirpich method.

The model was assembled as follows:

- The sub-basins were declared and the model was fed with the required parameters.
- It was assumed that the design rain is homogeneous throughout the basin.
- The histogram was introduced for design storms of 20, 50 and 100 years of return period.
- The SCS method was selected to calculate infiltration losses and the kinematic wave method to determine flood hydrographs.
- For the traffic of avenues, the kinematic wave method was also selected.
- Runs were made for the different return periods.

Obtained the results of the runs, it was compared with the preliminary analysis of the study carried out in the meteorology and hydrology section.

7.6. Results of the Runs of the HEC-HMS Model of the Cocolí River Basin:

In Table No. 3, the results of the runs carried out for the Cocolí river basin and sub-basins up to the diversion site are presented for return periods of 20, 50 and 100 years of return period.

Design flows to the Cocolí river diversion site were estimated for return periods of 20, 50 and 100 years at 250, 270 and 300 m^3/s respectively. It was observed that the flows estimated by the HEC-HMS, exceeds by 11% those obtained by the preliminary analysis, so the estimate is considered acceptable.

Table 4 shows the values of the hydrograph to the site of the diversion of the Cocolí River and in Figure 13 its respective output hydrograph.

Additionally, in order to verify the estimates of the design flows, an inspection and survey of the Cocolí river basin, in the lower part, was carried out in order to try to estimate some flood by means of the slope area. On June 12, 2007, the sections in the main channel of the Cocolí River and another in one of the main tributaries were surveyed in order to estimate a capacity.

The flows of the sub-basins of the small streams that will discharge to the channel of diversion of the Cocolí River, were determined by means of the rational approach method.

Table 3 shows the results of the run with the HEC-HMS for a return period of 100 years. It can be seen that the maximum value is 291 m^3/s, but for the run with the HEC-RAS, it was rounded and 300 m^3/s was used.

Table 3. Results by Subbasin of the HEC-HMS Model Runs for the Cocolí River for a Period of Return of 100 Years.

Hydrologic Component	Drainage surface (km^2)	Maximum Discharge (m^3/s)	Time to[3] Peak (hour)	Volumen (mm)
Cocoli 3	9.82	67.90	12Jul2003, 12:20	65.63
Cocoli 4	16.32	136.70	12Jul2003, 11:55	65.96
Cocoli 5	20.34	183.90	12Jul2003, 11:50	65.00
Cocoli 7	22.69	211.00	12Jul2003, 11:45	64.60
Cocoli 8	27.86	271.40	12Jul2003, 11:50	64.30
Output	29.51	291.40	12Jul2003, 11:45	64.58
Subcuenca 1	6.64	39.70	12Jul2003, 12:35	65.49
Subcuenca 2	3.18	28.40	12Jul2003, 12:10	67.13
Subcuenca 3	1.85	16.00	12Jul2003, 12:10	61.10
Subcuenca 4	4.02	62.80	12Jul2003, 11:35	62.24
Subcuenca 5	2.35	38.90	12Jul2003, 11:30	62.33
Subcuenca 6	3.11	36.60	12Jul2003, 11:50	61.81
Subcuenca 7	4.65	62.10	12Jul2003, 11:45	69.15
Subcuenca 8	2.05	37.80	12Jul2003, 11:25	69.36
Subcuenca 9	1.65	30.8	12Jul2003, 11:25	69.40

Table 4. Maximum Estimated Flows for the Cocolí River to the Derivation Site and for the Affluent that Intercept According to the Period of Return in Years

	Subbasin	Área (km^2)		Return Period in years (Tr)					
River/creek	#	Partial	Total	20		50		100	
				Q. Parcial	Q. Acum	Q. Parcial	Q. Acum	Q. Parcial	Q. Acum
Cocolí	1	6.64	6.64						
Cocolí	2	3.18	9.82						
Cocolí	3	1.85	11.67						
Cocolí	4	4.02	15.69						
Cocolí	5	2.35	18.04						
Cocolí	6	3.11	21.15						
Cocolí	7	4.65	25.80						
Cocolí	8	2.05	27.85						
Cocolí	9	1.65	29.50						
Cocolí			29.50	250	250.0	270	270.0	300.0	300.0
Victoria 4		0.65	30.2	10.0	260.0	12	282.0	14.0	314.0

[3] The date was taken arbitrarily, considering as a reference a rain event of the year 2003.

Victoria 3		0.14	30.3	2.5	262.5	3	285.0	3.5	317.5
Victoria 2		1.87	32.2	24	286.5	26	311.0	32.0	349.5
Victoria 1		0.49	32.7	6	292.5	7.5	318.5	9.0	358.5
Velásquez		12.90	45.6	90	382.5	100	418.5	110.0	468.5

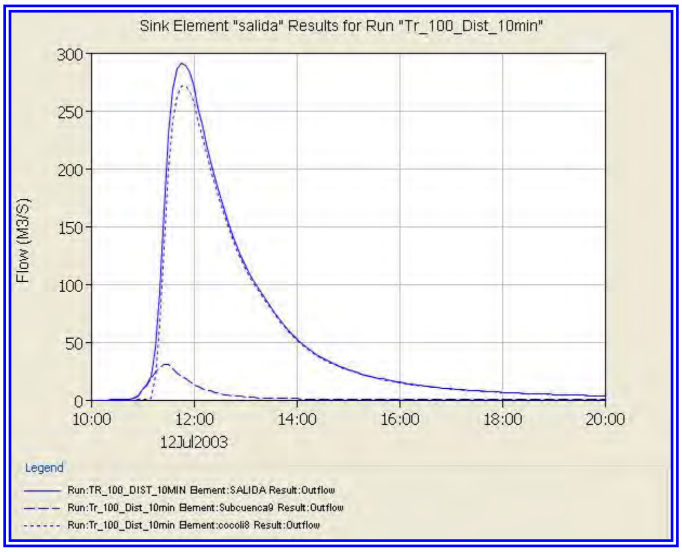

Figure 13. Output hydrograph of the Cocolí River to the derivation site for a return period of 100 years.

8. HYDRAULIC MODELING TO DETERMINE THE WATER PROFILES OF THE DIVERSION CHANNEL

To determine the water profiles along the route of the diversion channel of the Cocolí river, for the different selected return periods, the hydraulic modeling program HEC-RAS was used, which has been developed by the Hydrological Engineering Center of the United States Army Corps of Engineers, which has as its predecessor the well-known HEC-2 and has been considerably improved since its appearance in the early 60's.

The current version of the program allows calculations of water profiles for permanent and non-permanent flow in one dimension, sediment transport analysis of the bed and analysis of water temperature.

The HEC-RAS was selected as it is a public domain program, it is widely tested and has literature available for consultation.

The model has among its main features the modeling of water profiles along a channel or channel, modeling and hydraulic calculation of hydraulic structures such as bridges, culverts, etc, in addition to having a module that allows the design Hydraulic channels and calculation of cut and fill.

In the updated version of the HEC-RAS, the HEC-BETA, its applications are only limited by the imagination of the modeler.

9. MODEL REQUIREMENTS:

After selecting the model, we proceeded to study its minimum requirements. The information needed for the modeling included the topographic maps of the area, the surveying of the cross sections of the channel alignment and on-site inspections to evaluate all the existing structures. All in order to have a comprehensive representation at the time of the final design of the area that will cross the diversion channel of the Cocolí River.

The cross sections were supplied by the Topography and Topography Section of the Engineering Division, at the request of the Geotechnical Section, which made the preliminary layout of the Cocolí DCP. The sections of the channel were raised following the preliminary stations layout of 20.00 m and width of 50.00 m on each side of the center of the channel proposed.

Once the survey was carried out, the cross sections were sent to the Geotechnical Section for review and conversion of X, Y and Z coordinates, distance in meters. In Figure No. 19, the initial scheme mounted on HEC-RAS of the diversion channel of the Cocolí River is presented.

A total of 186 cross sections were provided for the DCP of the Cocolí River, for the Victoria Sur creek, nineteen cross sections for a length of 473.50 m and the Victoria Norte, five cross sections for a length of 175.00 m. The Velázquez River was used a total of 41 cross sections for a length of 820.00 m.

10. HEC-RAS MODEL RUNS

Obtained the design flows of the Cocolí river in the diversion site for periods of return of 5, 20, and 50 and 100 years, we proceeded to assemble the hydraulic model HEC-RAS.

The cross sections for the main channel and tributary streams were loaded, and the design flows for the selected return period were introduced, which in our case is 100 years. To connect the tributaries to the DCP of the Cocolí River, the junction function of the HEC-RAS was used.

Assembled the model, we proceeded to the design of the 3500 meter of the channel and the inflow entrances that will be affected by the future construction of the diversion channel.

In the preliminary runs only the main channel was designed, which was an iterative process. Subsequently, the tributaries were included and the respective runs were made, the outputs were analyzed and if problems such as channel capacity or high speeds arose, they were corrected immediately, and so on until the modeling of the channel was completed.

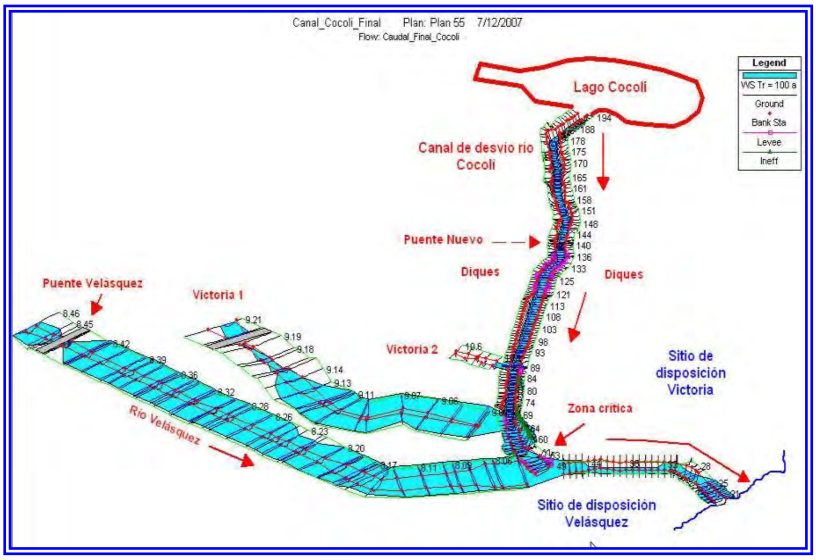

Figure 14. Initial outline of the diversion channel of the Cocolí River mounted on HEC-RAS.

After analyzing the capacity of the channel, the water profiles, the speeds and the slopes, we proceeded to declare the structures, in this case the preliminary design of the new bridge over the diversion channel, the culverts of the Victoria Sur and Norte creeks, Velázquez river bridge, with the purpose of studying its behavior and mainly determining the Extraordinary Maximum Waters Level in the bridge, which is a requirement of the Ministry of Public Works when a plan is going to be submitted for approval.

10.1. Problems Found:

The problems that arise from the successive runs of the model and the integration of the components during the design phase, the verification of the hydraulic design of the channel and the different restrictions and objections presented by the different units of the ACP that can be affected in some way or another. Some restrictions included the final alignment of the channel so that it does not affect protected areas, does not affect the slopes of the disposal areas, the channel floor and elevation of the bridge in the designed bridge area, excessive speeds, etc. A brief description of the problems encountered is presented below.

10.1.1. High speeds:

In the section between the start of the diversion channel and the projected bridge (station 0K + 00 to 1K + 00) high speeds are presented. This problem is caused because in a stretch of approximately one kilometer where you have to descend 4.75 m which creates high speeds due to the high slope and cause supercritical flow.

10.1.2. Channel capacity:

The outflows of water profiles of the preliminary runs, when the design flow was applied for a return period of 100 years, showed that in the section comprised between 1 k + 060 meters (taking as reference the beginning of the detour), water under the proposed bridge, water spills down the plains lateral to the channel. This is a product of the channel crossing a topographic zone of low elevation (flat plains).

10.1.3. Quebrada Victoria Norte channel not defined:

The exits indicate that the water spreads through the floodplain adjacent to the channel in that section, which may cause damage to the diversion channel.

10.1.4. Quebrada Victoria Sur presents backwater effects:

In this union, the channel of the stream is lower than the channel. This situation causes water from the channel to escape into the creek, flooding the natural floodplains of the creek.

10.1.5. Río Velázquez effects of backwater at the confluence with the diversion channel:

Due to the introduction of the diversion channel in the natural channel of the Velázquez river, which alter the natural conditions of the area, which is why this is considered one of the most critical points of the design due to three situations that arise: abrupt of the horizontal alignment (curve), the low slope of the natural course of the river and that the reach of the cross sections was short, because it only covered 20 m from the center of the river to each bank.

These situations cause turbulence and backwater effects at the confluence, which is why the particular situation of this point required a deep analysis. The initial outputs indicate that only by applying the design flow to the diversion channel (not the river), the water traveled upstream of the Velázquez River. The situation became more critical when the design flow was applied to the Velázquez River, since the water by the backwater effect flooded the plains upstream of the bridge.

This situation can cause problems due to the fact that upstream is the bridge over the Velázquez River and the development of the highly valued residential complex, "Tucán Country Club" which includes its own golf club.

12. RESULTS AND NECESSARY SOLUTIONS AND WORKS:

Due to the topographic configuration of the area where the diversion channel of the Cocolí River is projected, the following works will have to be carried out.

- At the confluence of the Cocolí River with the lake, it is necessary to contemplate the dredging and formation of a channel to channel the waters of the Cocolí River to the diversion channel.
- The curves should be smoothed, the unusable materials removed from the channel and the obstacles present in the area eliminated.
- At the mouth of the bypass, the unusable material should be removed and replaced or replaced by a more cohesive material and contemplate the treatment of the bypass mouth by means of rock, riprap, etc., since the turbulence associated with the change of slope and address can cause scouring problems later.
- Dam levee construction at least 17.00 m high in the lake to divert the waters of the Cocolí River to the diversion channel.
- First section of the channel includes from station 0K + 00 to 2K + 030.61. The first section starts at an elevation of 10.87 m PLD and ends before the confluence with the Victoria 2 ravine at 4,343 m PLD. The proposed channel is of trapezoidal section with slopes from 2 to 1 beginning with a width of 15.00 m and ending with 30.00 m.
- The structures of this section are the bypass entrance, four steps of 0.75 m in height to dissipate the energy, a reinforced concrete bridge of 11.00 wide by 36.00 m long and lower beam level of 10.52 m PLD.
- Before the construction of the bridge, it is necessary to demolish the reinforced concrete drainage system of dimensions 3.00 x 3.00 m in station 1 k + 013.89.
- At station 1 K + 017.00, replacement by a bridge with an elevation of the lower beam of the bridge of 1.80 m according to MOP requirements on the Maximum Extra Waters Level (NAME) of 8.72 m PLD.
- From station 1K + 080.00 to 1 K + 440.00, due to the topographic configuration of the land, it is required to build approximately 360 meters of containment dikes on both sides with an average height of 1.80 m, in order to contain the waters inside the channel.

- The second section of the diversion channel includes station 2 K + 060.00 to 2 K + 520.00. Within this stretch are the Victoria Norte and Victoria Sur gulches.
  - At the beginning of this section it is necessary to extend the section of the channel from 25.00 m to 30.00 m, along a segment of 20 m to reduce the backwater effect and allow access to the Victoria Norte creek.
  - At the end of the section at station 2 K + 520.00, channeling and construction of dams is required to channel the Victoria Norte ravine to the channel. A trapezoidal section with slopes 1: 2 and a length of 86.00 m is required.
  - In the Quebrada Victoria Sur it is required to cover the confluence with the diversion channel. Trapezoidal channel 2: 1 and width of the channel of 4.00 m. It should be improved with channeling from 214 m upstream of the junction with the diversion channel. Also in this section the profile with a slope of 0.0036 m / m must be improved.
  - In both cases, it is recommended to cover with rock, riprap or another type of coating to avoid the scouring of the entrances of the streams at the confluence with the channel.
- The third section, channeling and expansion of the Velázquez River channel 310.00 m upstream of the confluence with the new channel. Conformation of the entrance to avoid flooding the floodplains of the Velázquez River due to the introduction of the diversion channel of the Cocolí River.

The work of channeling and construction of the Velázquez river dikes will depend on the use given to the adjacent lands. If no significant use is intended, the construction of dams is not necessary.

13. CONCLUSIONS:

- Due to the lack of discharge measurements in the Cocolí river basin, indirect methods were used to estimate the design flows up to the diversion site and in the tributaries that will be affected when the diversion channel had built.
- To carry out the estimates of the design flows, the method of rational approximation, the Regional Analysis of Maximum Floods and finally the HEC-HMS were used in a preliminary manner.
- Design flows using the HEC-HMS were estimated using the "Alternate Block Method" and the flood routing for each sub-basin using kinematic wave method.
- The design flows up to the diversion site of the Cocolí River, have been estimated for return periods of 20, 50 and 100 years in, 250, 270 and 300 m^3/ s respectively.
- The design flows of the Victoria Sur creek to the confluence with the proposed diversion channel have been estimated for return periods of 20, 50 and 100 years in, 6.00, 7.50 and 9.00 m^3/s, respectively
- The design flows up to the Victoria Norte creek to the confluence with the proposed diversion canal have been estimated for return periods of 20, 50 and 100 years in, 24, 26 and 32 m^3/s, respectively.
- The design flows of the Victoria 3 creek to the confluence with the proposed diversion channel have been estimated for return periods of 20, 50 and 100 years in, 2.50, 3.00 and 3.50 m^3/s, respectively.
- The design flows of the Victoria 4 creek to the confluence with the proposed diversion canal have been estimated for return periods of 20, 50 and 100 years in, 10, 12 and 14.00 m3 / s, respectively.
- The design flows of the Velázquez River to the confluence with the proposed diversion canal have been estimated for return periods of 20, 50 and 100 years at 90, 100 and 110 m3 / s, respectively.
- The design flows of the diversion channel of the Cocolí River until its discharge at the entrance of the Pacific channel, have been estimated for return periods of 20, 50 and 100 years in, 382.5, 418.5 and 468.5 m^3/s, respectively.
- For the hydraulic modeling of the water surface of the diversion channel, the HEC-RAS was used, a program developed by the Hydrological Engineering Center of the Engineers Corps of the United States that is free of charge.
- The model was fed with 176 cross sections for the diversion channel of the Cocolí River for a length of 3480.782 m.

- For the Quebrada Victoria Sur, it was fed with 19 cross sections for a length of 473.50 m and the Victoria Norte, 5 cross sections for a length of 175.00 m. The Velázquez River was used a total of 41 cross sections for a length of 820.00 m.
- The level of extraordinary maximum waters at the site of the new bridge for a return period of 20, 50 and 100 years respectively is 8.39, 8.53 and 8.72 m PLD, respectively.
- From station 1 K + 80 to station 1 K + 440.00, due to the existing topographic conformation, dykes are required to contain the water inside the diversion channel.
- At the confluence of the Velázquez River with the new diversion channel of the Cocolí River, accessory works are required (60.00 m wide pond) with the purpose of reducing the turbulence and the backwater effect that occurs when the river joins the river Velázquez with the channel. The Velázquez River must be channeled at the confluence with the new diversion channel.
- It is required to enable the entrances of the Victoria Sur and Victoria Norte creeks.
- Depending on the outputs of the HEC-RAS, it is required to excavate approximately 442 192 m^3 of material.
- For the new bridge the Extraordinary Maximum Water Level (EMWL) for a return period of 100 years is 8.72 m PLD, so the lower beam level should be 10.52 m to have a clear distance of 1.80 m to meet the MOP specifications.
- For a return period of 50 years the Extraordinary Maximum Water Level is 8.53 m PLD, so the lower beam level must be 10.33 m to have a clear distance of 1.80 m to comply with the MOP specifications.
- In the section of the new bridge over the diversion channel from station 1K + 006.00 to 1K + 040.00 the slopes will be 1: 1, with a floor of 20.00 m wide and will be covered both in the bottom and in the slopes. This is the product of the 24 inch IDAAN pipeline which is recommended not to move it. From station 1K + 040.00 to station 1K + 050.00 the canal slopes are returned to 2: 1 with a floor of 25.00 wide and must also be coated.
- The average water level expected in the bypass channel for a 5-year return period design is 2.00 m above the channel bottom.
- For a return period of 100 years, the edge condition used at the end of the channel was for a tide of 3.04 m PLD.

14. RECOMMENDATIONS:

- It is recommended to validate the estimated data up to the diversion site of the Cocolí river diversion channel project through the installation of a temporary measurement system upstream to confirm the goodness of the estimates.
- Due to the low slopes, once the construction of the Cocolí River diversion Channel has been completed, from the new bridge to the mouth in the sea, it is necessary to give it periodic maintenance to avoid the growth of vegetation in the channel.

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15. HEC-HMS AND HEC-RAS OUTPUTS:

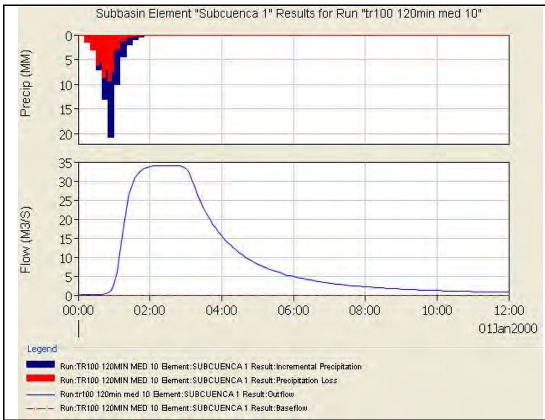


Figure 15. Hydrograph subbasin 1.

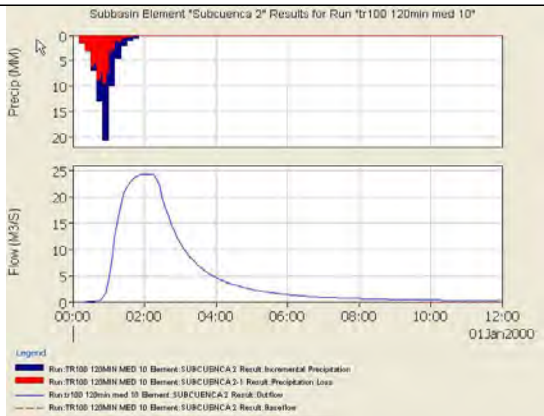


Figure 16. Hydrograph subbasin 2.

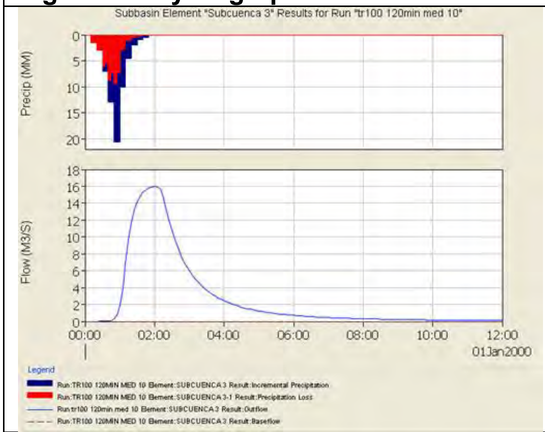


Figure 17. Hydrograph subbasin 3.

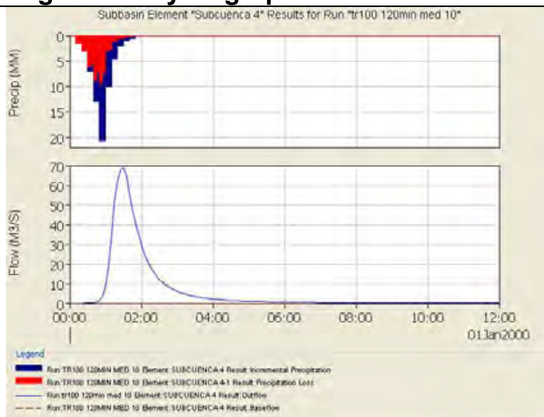
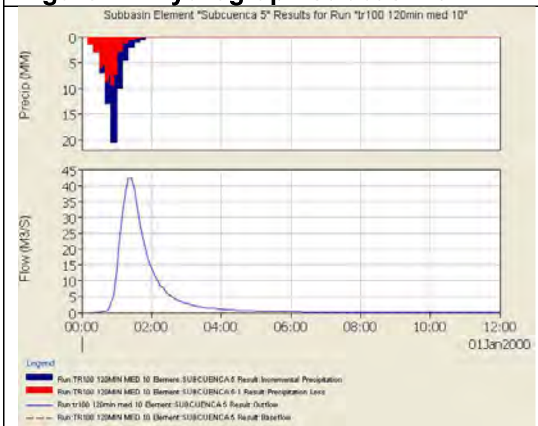


Figure 18 Hydrograph subbasin 4.



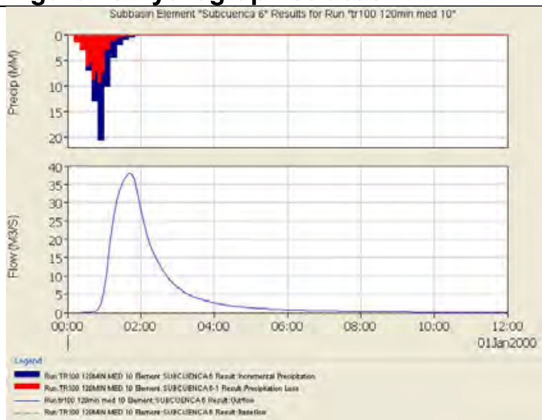
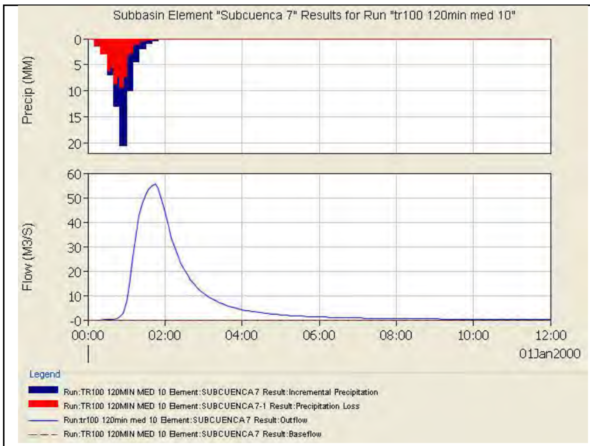


Figure 19. Hydrograph subbasin 5.	**Figure 20. Hydrograph subbasin 6.**



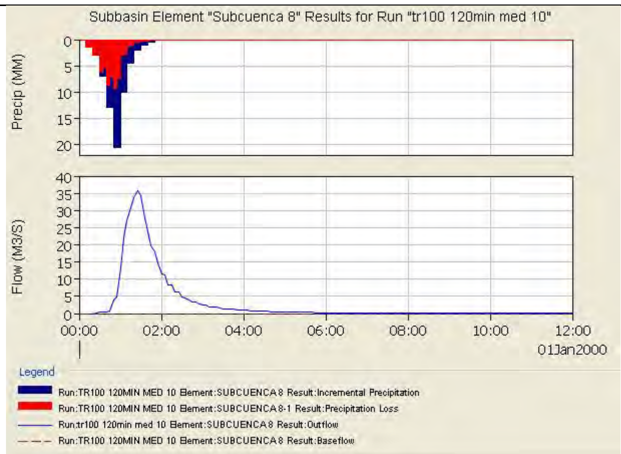
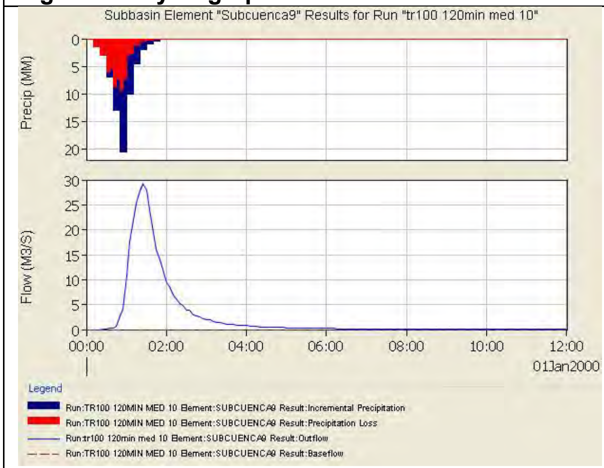


Figure 21. Hydrograph subbasin 7. **Figure 22. Hydrograph subbasin 8.**



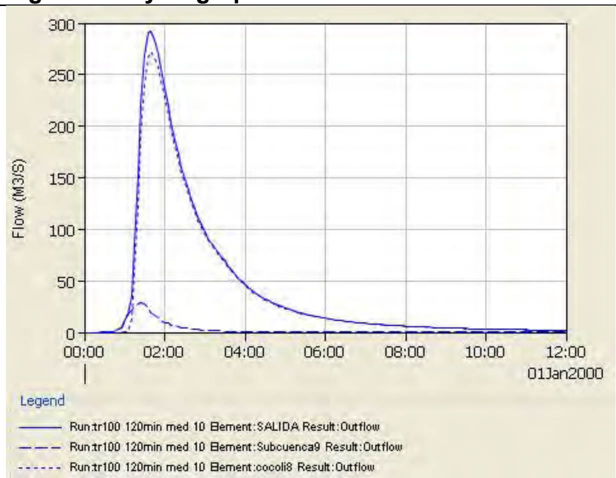


Figure 23. Hydrograph subbasin 9. **Figure 24. Total outflow for 100 years return period.**

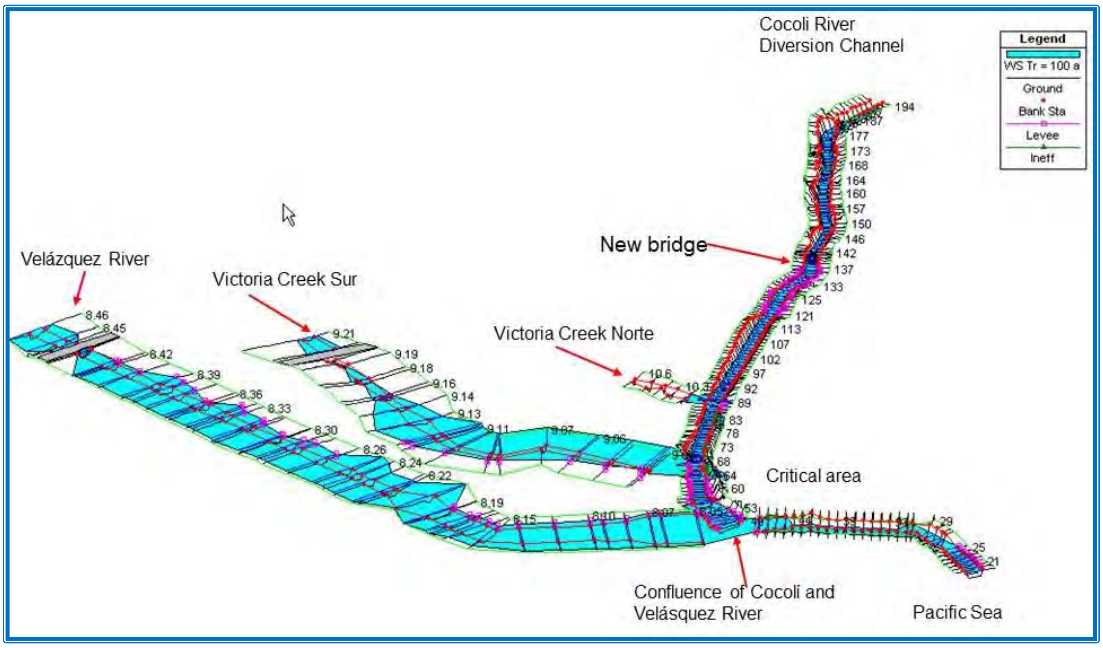


Figure 25. Scheme of the diversion Channel of Cocolí River.

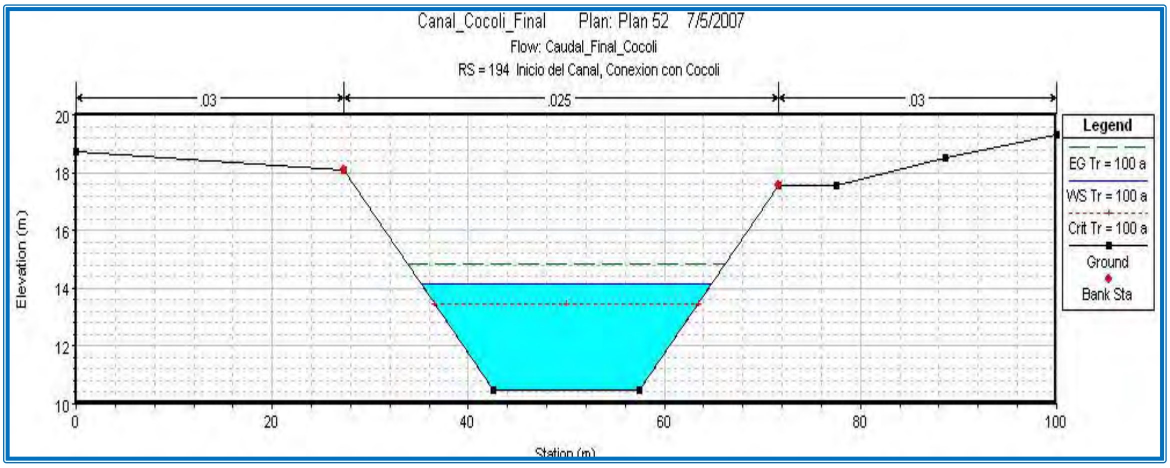


Figure 26. Prismatic section proposed for the diversion Channel.

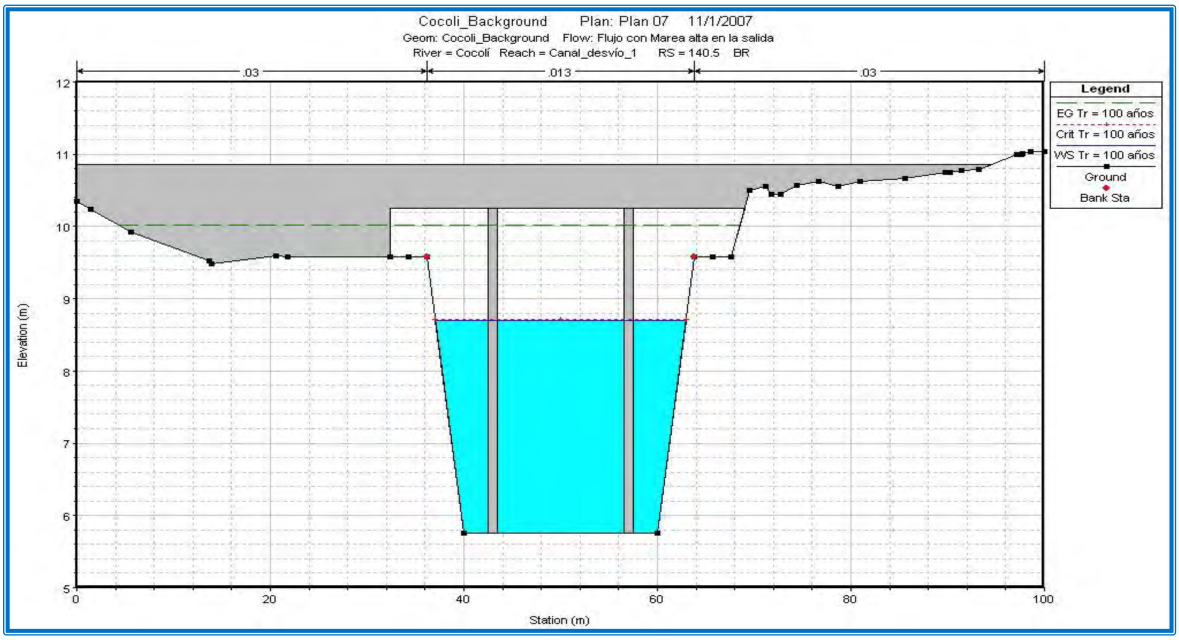


Figure 27. Extraordinary Maximum Water Level for the new bridge.

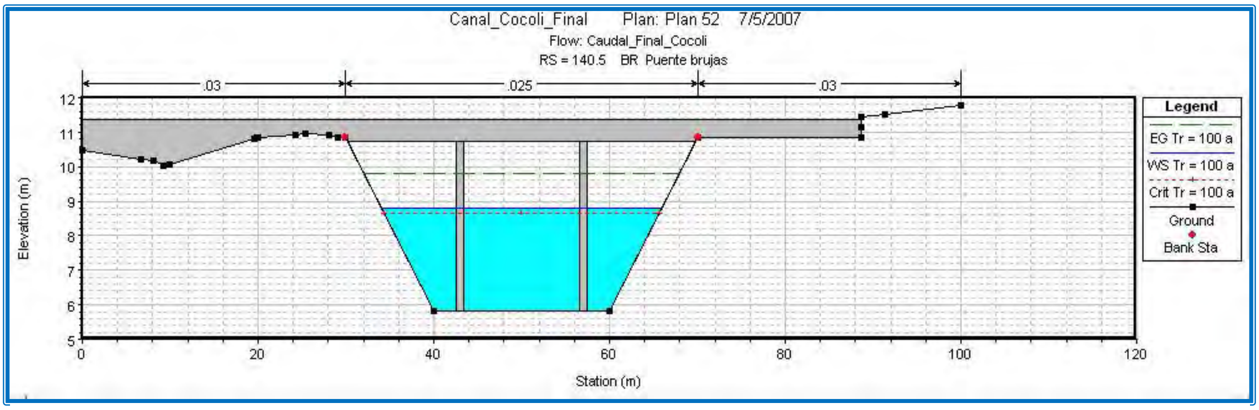


Figure 28. Extraordinary Maximum Water Level for the Brujas Bridge in Velázquez River.

Figure 29. Image of the Cocoli river deviation channel and the new locks of the Pacific side.

LENGTHENING OF QUESNOY-SUR-DEULE LOCK – DESCRIPTION OF A LATERAL SLIDE CONSTRUCTION

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LENGTHENING OF QUESNOY-SUR-DEULE LOCK – DESCRIPTION OF A LATERAL SLIDE CONSTRUCTION METHOD

by

P.Schalkwijk[1], S.Legrand[2] and C.Chéné[3]

ABSTRACT

The French Waterways Authority (Voies Navigables de France) manages in the North of France a particularly active territory in terms of river traffic. The Seine Nord Europe canal project, as a future link between Belgium and France, needs to improve two large-gauge waterways. One of these two itineraries, the canalized Deûle, presents a problematic lock on its route, the Quesnoy-sur-Deûle lock.

This lock is located in France, close to the city of Lille, and less than 5 km from the Belgian border. The problem stands in the fact that this lock has only one lock chamber of 110 m long, while all the others have lock chambers of 144 m. It therefore does not allow the navigation of Large Rhine boats of 135 m, which are very widespread in the Netherlands, Belgium and on the Rhine River. For that reason, BRL ingénierie was commissioned by VNF in 2013 to study the design of the extension of this lock to its new dimensions of 144.6 x 12 meters. The main difficulty for this operation is that the river traffic can only be stopped during a very short period, which should not exceed 2 weeks per year. The second difficulty is that the lock is located in a geological context of Flanders Clay with weak and very particular characteristics.

The proposed solution consists of constructing the walls of the new lock head set back from the chamber lock, under the shelter of two cofferdams and then putting them at their final location by a lateral slide operation. This operation offers the great advantage of providing, in fine, a conventional, reinforced concrete lock, without requiring long closure. However, it induces a certain number of problems to be solved, such as the construction of modular evolutive cofferdams, the placement of tracks and slide bearings capable of bearing loads around 3 000 t, the construction of homogeneous foundations, a phasing of works allowing to work isolated from water, the connection to the existing lock.

The manner in which each of these difficulties has been solved will be developed in the article. The aim of the article is to propose an unusual solution to the problem of the extension of locks with a limited cut-off of the river traffic.

1. INTRODUCTION

1.1. The situation of French waterways in Haut-de-France region

This lock of Quesnoy-sur-Deûle is located in France, close to the city of Lille, and less than 5 km from the Belgian border (**Fig.1**). It is located on the large gauge itinerary of canalized Deûle.
The project to lengthen the Quesnoy-sur-Deûle Lock is part of the reconfiguration mission for the Seine Nord Europe canal and is part of the modernization of the Seine-Escaut connection.
In the context of the increase in river traffic, the objective of the lengthening of the lock is to allow access to Va + units (CEMT ranking) which correspond to the large Rhine boats of 135 m. In addition, it will ensure the continuity of navigation to the large-gauge waterways network, as well as sustain the growing development of river transport.
Indeed, in the context of expected traffic developments with the Seine Nord Europe Canal, inland waterways will be increasingly used in the region (between river ports, major seaports and multimodal platforms) as well as externally to Paris region or to neighboring countries (Belgium and the Netherlands).

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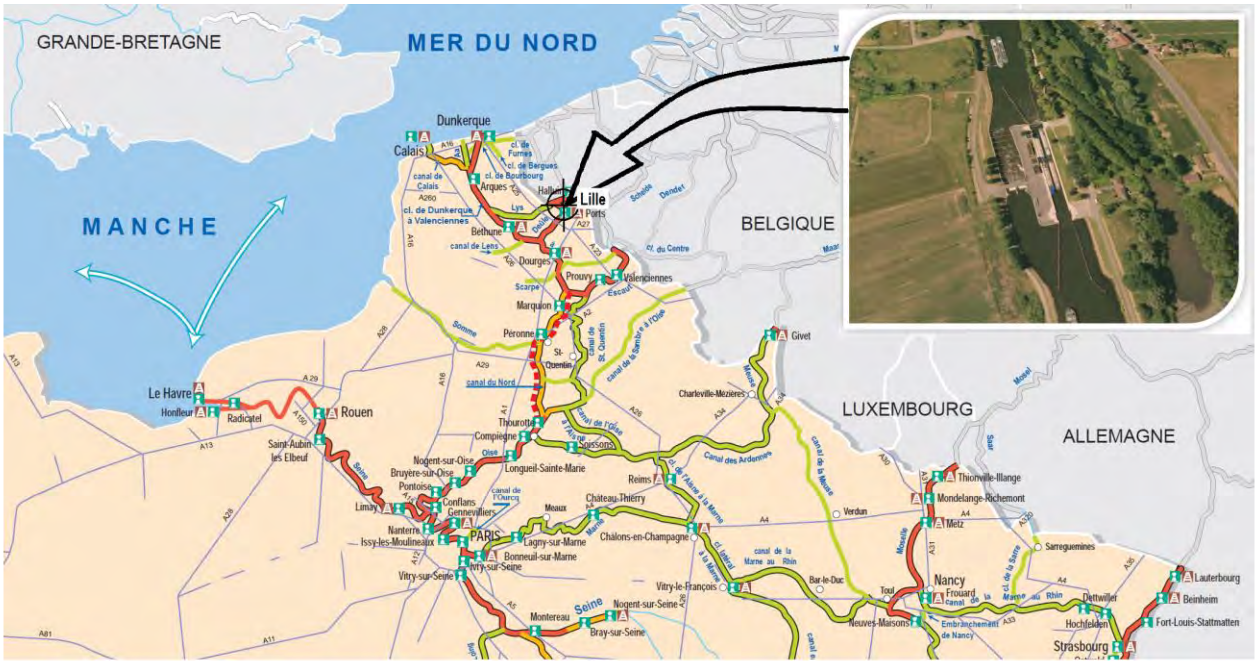


Fig. 1: Quesnoy-sur-Deûle lock site plan

1.2. Quesnoy-sur-Deûle Lock and its length chamber problem

Currently, the Deûle is 1350 t gauge (CEMT IV gauge) and a 3000 t recalibration (110 m long CEMT Va gauge) is in progress.

The lock of Quesnoy-sur-Deûle currently has a lock chamber of 110 m which is compatible with the current gauge but also with the project gauge of 3000 t.

However, as part of the projections for the 4400 t gauge (CEMT Va + - Large Rhine boats of 135 m) and the coherence of the route (the chambers of the other locks on the Deûle upstream of Don and Grand-Carré are 144 m long and the Comines chamber lock on the Lys downstream has just been lengthen to 185 m), a lengthening is necessary to unlock the lock created by Quesnoy-sur Deûle lock.

In a second step, the project of doubling the lock with the construction of a new lock of 190 m will be necessary (studies carried out up to the detailed project stage) by 2040. Thus all the installations related to the works of lengthening of the existing lock must be compatible with the future doubling of the lock.

2. CONTEXT

2.1. Geological context

The geological map of the area (map of BRGM at 1/50,000 scale of Lille and associated legend, **Fig.2**) indicates the following lithology:

- Superficial formation (AO): Modern Quaternary Alluvium (green)
- Alluvial formation (AD): Limons of the plain of the Lys (orange stains)
- Yprésienne formation (AR): Clays of Flanders (orange stripes)
- Landenian formation: a sandy facies with a clay facies base

- Senonian formation: white chalk

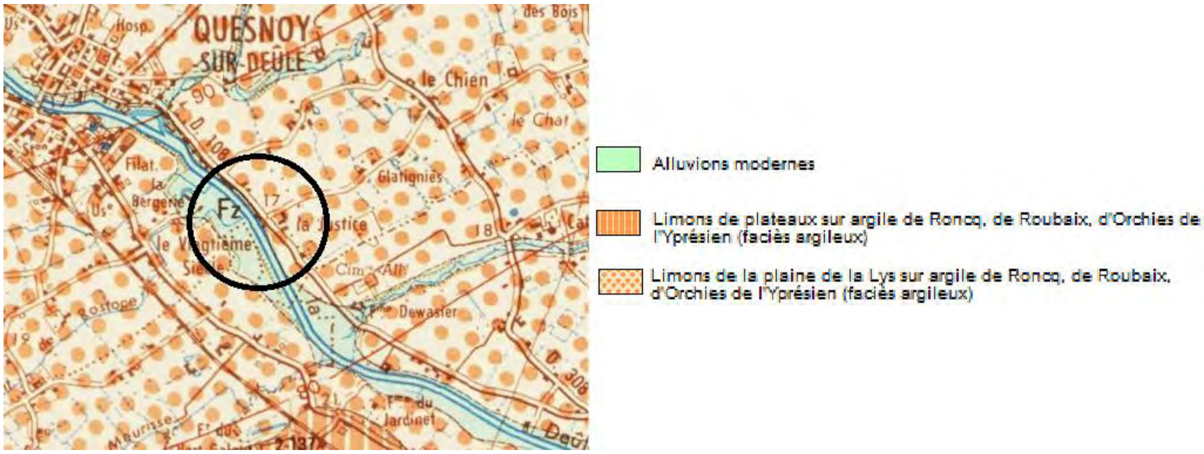


Figure 2: Extract from geologic map of Lille at scale 1/50 000th and associated legend

The tops of the Landenian and Senonian formations are respectively around 35 m and 80 m deep. They were not encountered during reconnaissance. The base of the foundations of the structure is therefore located in the Ypresiennes formations which are very soft clays and which can be very steep whose top is variable (here, between 7 and 12 m of depth).

The superficial (AO) and alluvial (AD) formations are very heterogeneous: a mixture of sands, silts and clays sometimes containing organic matter (for modern alluvium) and relatively poor mechanical characteristics.

2.2 Hydraulic context

At Quesnoy-sur-Deûle, the median flow is 10.3 m^3/s. The navigation has to be possible in normal conditions for the minimal flow rate of 1.72 m^3/s. This flow has to ensure the supply of locks and works linked.
The dams on the river maintain a constant water level. Next to the lock Quesnoy-sur Deûle lock, there is a dam that maintains the upstream levels at 14.72 mNGF. However, the downstream level is not constant as the next dam is far. The target level of the downstream reach is therefore 11.25 mNGF, representing a 3.47 m height drop. Navigation here is allowed up to a 50 m^3/s flow rate.

2.3. River traffic

Today, 13 000 ships per years including 9 000 fully-loaded ships cross the locks. This traffic represents 5 million tons of transported goods. We noted that the annual number of ships has decreased in the past few years, but the annual transported tonnage remains stable; this clearly indicates the growth in ship capacity. In addition, Netherlands and Belgium use a lot of river transport and France projects to develop naval transport in the North. As such, the Quesnoy-sur-Deûle lock is a sticking point as it is the smallest lock on this axis. It limits the length and the number of ships that can pass this channel. Traffic projections suggest that the lock will be overloaded in 2020-2025. This lengthening project is therefore intended to bring the lock to the standard of the others infrastructure on this axis.

2.4. The existing lock

The site of the lock consists of a lock and a dam (**Fig.3**).

The lock. The lock was built in 1980. It has a chamber of 110 m x12 m. The minimal anchorage is 4.30 m. The current structure is "U" shaped, in reinforced concrete. It is about 133 m long. The lock walls are 8.96 m high. The upstream head is equipped with miter gates located on breast wall at 3.75 m height. Two aqueducts, which surround the gate, assure the filling of the lock. Their sections are 2.5 m x2.5 m. They are equipped with two slice gates. The downstream head is equipped with miter gates which have 8 valves to ensure the emptying of the lock. Currently, the filling and the emptying of the lock lasts 5 to 6 minutes. Then, the ship crossing lasts about 19 minutes including ship maneuvers, gates and valve maneuvers and filling or emptying.

The dam. On the left side, a check dam, rebuilt in 2009, composed with 2 passes, has a discharge capacity of 125 m^3/s.

The median embankment. It is located between the dam and the lock. Three premises are built on it: the technical building, the command building and the archive building.

The right bank. Two houses belonging to the site are on the right bank.

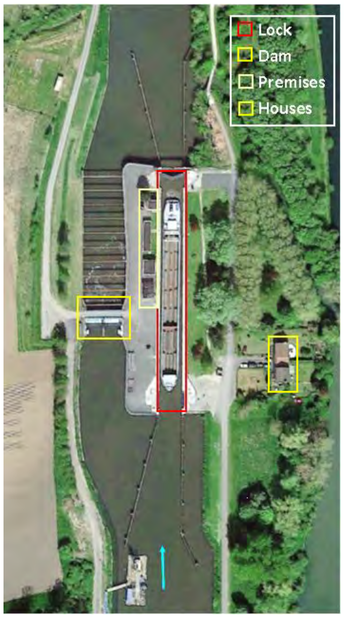


Figure 3: Location of the main facilities on the site

3. THE NEW LOCK

The project consists of the following elements: the lengthening of the lock (**Fig.4**), a fish pass, a pumping station, 2 ship parking areas (upstream and downstream), recalibration of the canal close to the lock in order to have an anchorage depth of 3.50m (760m on upstream reach and 340 m in downstream reach), replacement of guide rails, and refurbishment of three premises.

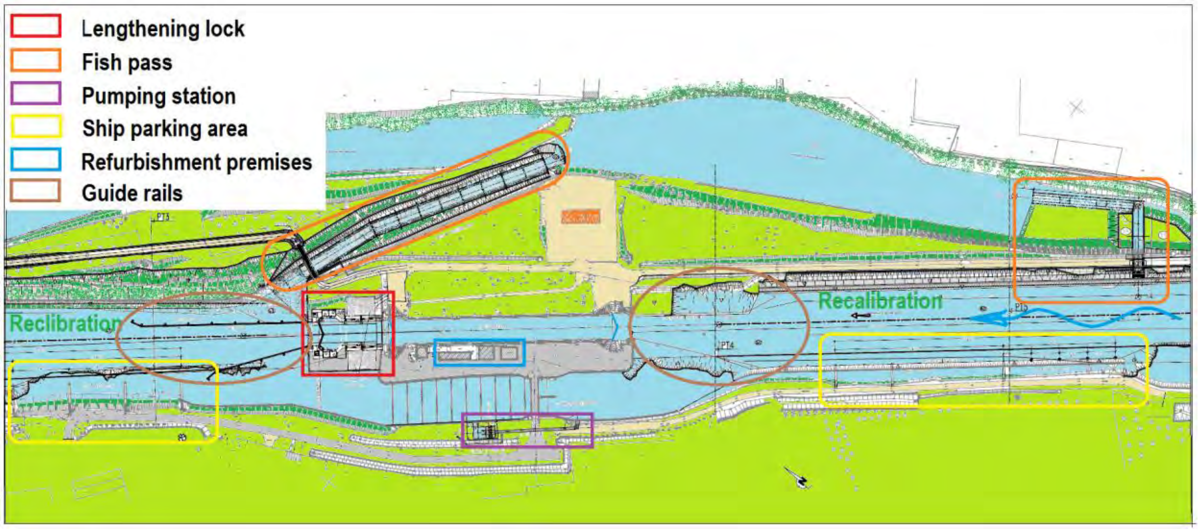


Figure 4: Plan view of the lock site

Only the lock lengthening is described below.

The lengthening of the lock is to take place on the downstream side of the lock to work with lower water level and reduce hydraulic strength, and to avoid deconstruction of the breast wall. The aim of the lengthening is to obtain a usable length of about 144.60 m. Then, the total length of the extension is 39.55 m (**Fig.5**). The shape and the material of lengthening will be identical to the current one: “U” shape (**Fig.6**) in reinforced concrete. It is superficially based on the clays of Flanders.

The common parts (lock walls, invert) and the head block are connected by dimpling system with WATERSTOP sealing strips. These systems allow the independent slipping of the blocks, a mechanical connection which avoids misalignments between blocks, and structural independence which allows shrinkage of concrete without cracks.

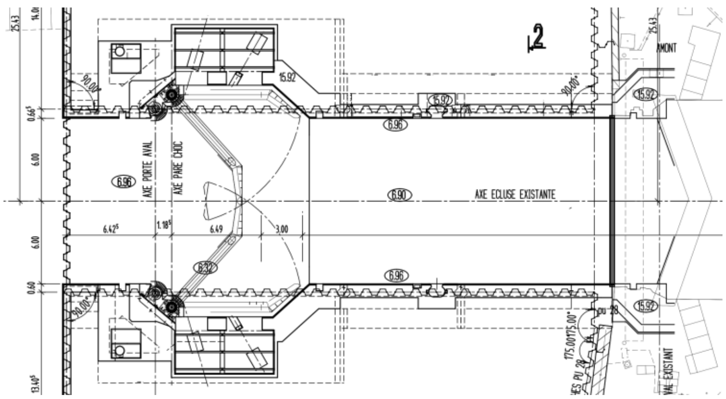


Figure 5: Plan view of lock extension

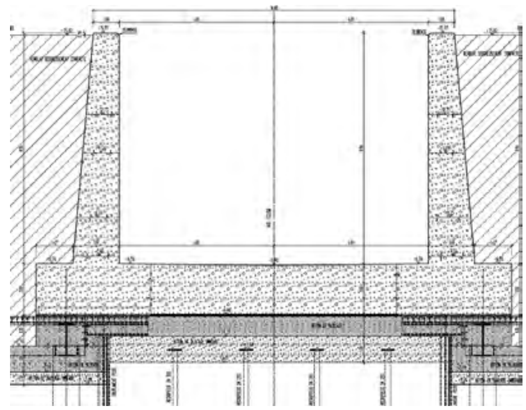


Figure 6: Cross section of the lock extension

The downstream head is equipped with a 20° angle miter gate bearing 6 valves, 3 per leaf, totaling a section of 11.52 m², equal to the current section. The gate is 65 cm higher than the upstream water level and a 1.2 m width walkway is located on top of the gate at embankments level. The actuators are electromechanical cylinders. Their capacity is 20 t and their extension length 2.6 m.

Gate are protected by bumpers. They are composed of 2 pivotal elements that behave more or less like a small gate. Each element is linked to a gate leaf by chain. Then, the gate movement drives the bumper movement. A Jarret damping system equips each bumper leaf. Bumpers can absorb energy of shocks up to 280 KJ.

Guide rails are placed downstream on the lock. They are composed of piles and rails.

The new lock, as described above and with the same valve section as the existing structure, raises the washing out time of the chamber by approximately one minute. The new lock will thus be able to perform a maximum of 41 cycles per day, three less than its current ability.

The projected total for the project is about € 30 M, including tax.

4. CONSTRUCTION DETAILS

4.1. Evolutionary cofferdam construction and construction phasing

The implementation of this extension raises several difficulties. The first is that the average completion time for a traditional reinforced concrete structure like this one is estimated at 4 months. This implies cutting off navigation for 4 months. For the operator, navigation cannot be stopped longer than for a few weeks. It is therefore necessary to find a way to allow construction at the same time as allowing boats to cross. As a matter of fact, the lock walls do not restrict the navigation. However, their construction requires a dry environment, sheltered by a cofferdam. It also requires protecting the workers in the cofferdam against shocks from boats. These constraints require the construction of the basements with a withdrawal of at least 0.70 m from the bare walls of the locks already built (which are spaced 12 m apart). Two solutions are available to the designer: either leave these newly constructed walls in place, or bring them in line with the previous ones. The first solution, although simpler to build, requires the design of a wider door and bumper, as the opening increases from 12 to 13.4 m. In addition, it would create jutting angles in the wall, which would be potential sources of shocks to the boats.

The second solution, presented here after, was therefore chosen:

- in a first step, to build the extension walls in withdrawal of approximately 3.50 m of the existing walls, sheltered by two small cofferdams, one for the left wall and one for the right wall;
- And, secondly, to move each wall - by sliding it on metal profiles - to its final location protected from the water by a large cofferdam uniting the two previous small ones.

These constructive principles led to the establishment of a phasing of work over 3 years including 3 closure periods of respectively 2, 4 and 3 weeks. It is described below

Work during closure period number 1

- Removal of guide rails of the downstream out port and demolition of the downstream slab;
- Partial earthworks in the channel to a depth of 2 m;
- Construction of two cofferdam (**Fig. 7**), one on each side of the channel;
- Each cofferdam has a rectangular plan-view shape of 37.5 m x 13.2 m; The 2 cofferdams are spaced 13.1 m apart; The sheet piles (PU28) have a height of 20.5 m;
- Implementation of micro piles at the chamber location according to a 2.75 m square mesh.

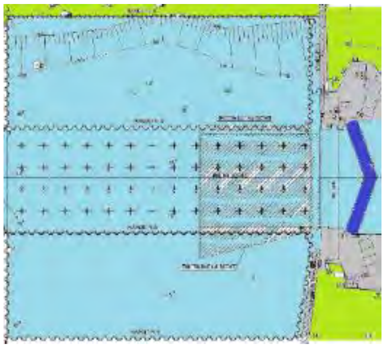


Figure 7: Closure period 1 - View of the two small sheet pile cofferdams

Work during the intermediate period between closure periods 1 and 2

This period has a projected duration of 11 months.

- 1st phase of earthworks in water over 5 m inside the 2 cofferdams;
- Driving of steel piles of the temporary support system;
- After dewatering, placing the walling, struts and tie rods at the lower walling, on the sides facing the banks;
- 2nd phase of earthworks under water (height 5 m), casting of a 0.50 m thick concrete layer and dewatering of the 2 cofferdams;
- Cutting of piles upper parts, setting of the temporary support structure and casting of a 1 m thick layer of concrete all around falsework;
- Lock wall construction (**Fig. 8**).

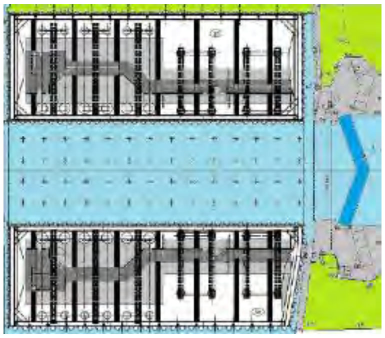


Figure 8: Works between closure periods 1 and 2 - View of the walls built back from those of the existing lock

Work during the closure period number 2

- Driving of 2 sheet pile walls to connect the 2 small cofferdams, one upstream and the other downstream;
- Partial dewatering and setting of I-shaped walling with variable height (0.90 m / 1.70 m) on both walls. Walling is held by 2 Ø800 struts.
- Earthworks under water in the central box;
- Casting of blocking concrete 1 m thick at the chamber emplacement & dewatering;
- Cutting of the intermediate sheet pile walls located on both sides of the chamber area and removal of associated walling and struts; obtaining a cofferdam of 37.5 m x 39.5 m;
- **Sliding of the walls: blocks 1 and 2 and head block;**
- Removal of sliding rails and implementation of concrete under the walls;
- Building of the central part of the invert;
- Building of the connection between lock blocks and sheet piles walls upstream and downstream;
- Filling of the new chamber; disassembly of the walling and struts Ø800 in the chamber;
- Cutting of upstream and downstream sheet pile walls in the chamber; Opening for navigation (**Fig. 9**).

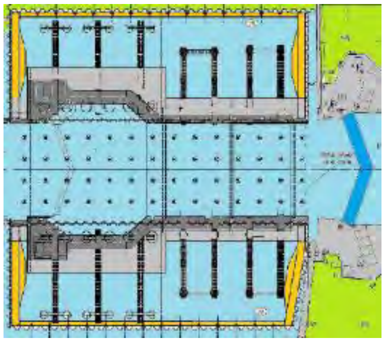


Figure 9: View of the work at the end of closure period 2

Work during the intermediate period between closure periods 2 & 3

- Finishing the lock walls and implementation of tie rods between left lock wall and the left bank sheet pile wall.
- Backfilling of the two small cofferdams (**Fig.10**)**.**

Work during the closure period number 3

- After setting maintenance cofferdams and dewatering, connection of the extension to the old lock.
- Equipment installation (gate, bumpers, operating devices);
- Filling of the chamber and opening for navigation.

Figure 10: View of the work at the end of the intermediate period 2/3

The phasing is summarized in the table next page (**Fig.11 to 19**).

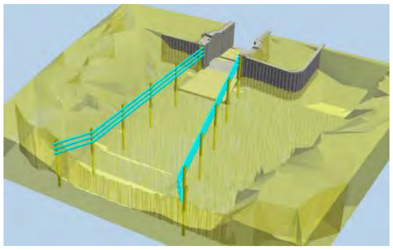


Figure 11 - Phasing - Stage 0

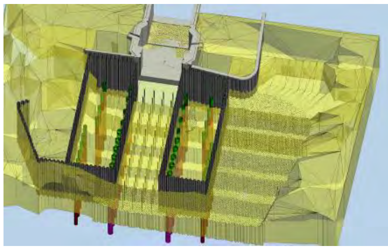


Figure 12 - Phasing – Closure period 1 – Smalls cofferdams and piles

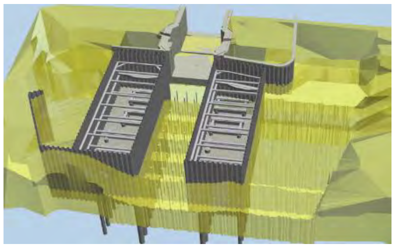


Figure 13 - Phasing – Intermediate period between closure periods 1 et 2 – Lateral slide tracks

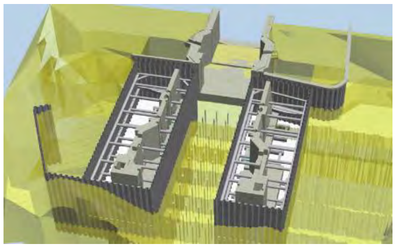


Figure 14 - Phasing - Intermediate period between closure periods 1 et 2 - Walls

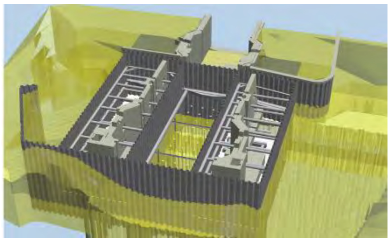


Figure 15 - Phasing – Closure period 2 – Unification of small cofferdams

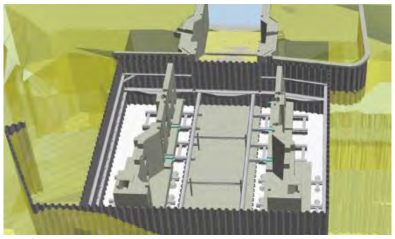


Figure 16 - Phasing - Closure period 2 – Realization of the large cofferdam

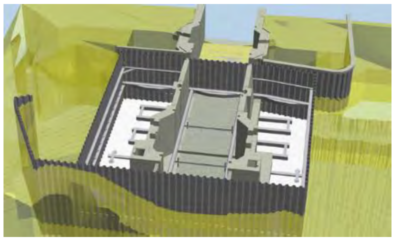


Figure 17 - Phasing - Closure period 2 – Sliding of the walls

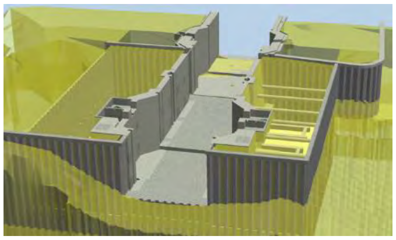


Figure 18 - Phasing - Intermediate period between closure periods 2 and 3 – Embankments

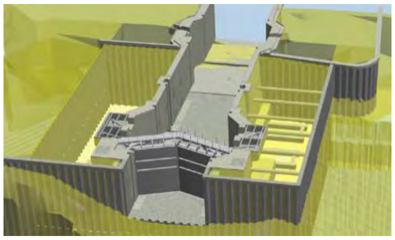


Figure 19- Phasing - Closure period 3 – Connection structure and gate

4.2. Implementation of sliding rails

Sliding structure

The large concrete slab at the bottom of the excavation is based on the clays of Flanders. The bearing capacity of these clays is too weak to withstand localized efforts during the shifting phase. This is why foundation piles are planned to support the beams of sliding.

For each line of wall, two rows of piles are planned: a line of front piles, at the limit of the lock chamber, and a line of piles behind the walls. The two lines are 9.2 m apart. These rows of piles are connected by I-shaped metal beams, 1.25 m high. These I-beams are laid on reinforced concrete plugs connected to the piles. These beams support the sliding rails. These are metal box girders of section 0.75 m x 1.25 m (b x h). Three sliding beams are planned for the head block wall and 2 beams of sliding for each current wall. A beam under the head block has to support a load of 1000 t. During construction, and up to the start of sliding, the mass of the walls is distributed more or less equitably between the two rows of piles. During sliding the mass of the walls will be carried toward the chamber lock. The latter supports more than 80% of the load at the end of sliding. This is why the piles of this row are designed to each support a load of 500 t. These are steel piles with a diameter of 1400 mm. For the line back of the lock chamber, we adopt piles of 250 t unit capacity and 1000 mm diameter. As the expected masses of the current blocks are much lower, 1000 mm diameter piles will be sufficient for both front and rear.

The beams are surmounted by sliding rails. These are beams made of 2 HEM 260 profiles welded side by side. The sliding of the lock walls is carried out on these sliding rails.

Sliding operations

Pushing is carried out on each rail by means of a jack fixed between the wall and the rail. The body of the jack is fixed on a metal shoe itself bolted to the rail. The cylinder head is attached to a metal bearing anchored in the concrete of the lock wall.

The walls lie on the rails via plates embossed on the underside and coated with PTFE (brand "Teflon"). The friction on the rails is therefore limited to about 15% at startup and 5% in sliding phase, which allows the use of double-acting jacks of 300 t capacity. The progress will be obtained by steps of 300 mm, all jacks pushing at the same time, according to the following cycle:

- The shoes are bolted on the rails, 300 mm translation of the lock wall by jacking out;
- Unbolting and translation of the 300 mm shoes by jacking in;
- Bolting of the shoes.

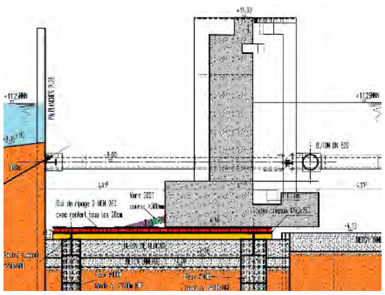


Figure 20: Lock wall in its final sliding position

The wall must be able to be vertically jacked at all times in order to allow the lubrication of the rails or the transversal adjustment of the load. For this purpose, 4 jacks per rail will be provided.

Setting on final bearings

At the end of sliding the remaining gap between the walls inverts and the concrete layer is filled with mortar (coarse concrete aggregate or lean concrete). The sliding rails and jacks are then removed. Spaces left empty are filled with mortar.

4.3. Arrangements to respect short closure periods

In the context of intense river traffic at the lock of Quesnoy-sur-Deûle, strong constraints are imposed to keep navigation closures to a minimum.

Thus, these closure periods should be reduced to 3 - spaced by about 1 year - limited to respectively 15, 30 and 20 days. Closures of 30 days being extremely restrictive for the operator of the waterway, the increase of the duration of closure beyond 30 days is not possible.

Thus steps have been taken to respect these time restrictions:

- Optimization of working days during closures with the inclusion of 6 days of activity per week and shift work whenever possible (demolition operations and earthworks in 3 shifts, piles driving operations in 2 shifts, but lateral slide and electricity operations in one shift).
- Performing a sliding test before the start of the 2nd closure period in order to avoid losing time during the actual sliding during closure.

- Definition of a point of no return for the key closure period, namely the second one, which consists in the possibility of taking a decision according to the actual progress on the 10th day of the period (i.e. the large cofferdam must have been dewatered by this date at the latest):
  - Either continue the work and complete the work within the time limit;
  - Either stop the work and return to navigation prematurely and lose the slot of closure expected. This alternative, even if anticipated, is obviously not desirable and will be the subject of dissuasive penalties.
- Consideration of margins respectively of 2, 3 and 2 days for closure periods 1, 2 and 3 in order to control a skid of the schedule.
- Development of a detailed program on a daily basis as early as the detailed project phase to ensure the feasibility of operations to be carried out under closure within the deadlines.

4.4. The clays of Flanders

Presentation of the issues

Feedback on the sector (construction of the dam on lock site in 2009 and implementation of sheet piles upstream of the lock on the left bank in 2015) and more generally on the region (different sheet pile construction sites) have shown that the completion of sheet piles driving and / or tie rod grouting work in the clays of Flanders could present risks in the construction phase which must therefore be anticipated.

The design of geotechnical structures (cofferdams, micro piles and grouted tie rods) allowing the lengthening of the lock to be completed therefore requires:

- For the cofferdam and the piles of the falsework: The implementation of relatively heavy profiles throughout a great height of Clays of Flanders. Indeed, the piles are 30 m long and the sheet piles (a linear of 275 ml) are 20 m long. The plastic and stiff nature of these soils induces a significant risk of refusal. The sheet pile profiles have therefore been chosen to safely allow their resistance to the forces generated during implementation.
- For tie rods: performing grouting in clay soil may become problematic because of the presence of sensitive structures located in the immediate vicinity (namely the existing lock). Indeed, the tie rods (28 in number) are 25 m in length, including 17 m of sealing in Flanders clays with risks of "breakdown" of the ground under the effect of grouting. This risk is even higher as the injection mode envisaged in the project is the “Repetitive and Selective Injection” which requires a good management of the injection pressure in order not to slam the clays. This is especially critical since clays have pressure limits relatively close to the injection pressures used for the implementation of this technique. However, since the tie rods were not executed under the existing lock, this risk remains relatively unlikely.

Management of the risk of refusal

In order to guard against the risks of a mismatch of the driving technique, the drill capacity or the choice of profiles with the geotechnical context, a preliminary driving test is therefore planned before the procurement of works to ensure the feasibility of the works envisaged at the project stage. This driving test will include the registration of a 30 m and 1000 mm diameter pile and 8 PU28 sheet piles (4 pairs). It will also make it possible to carry out vibration measurements on the existing lock in order to ensure there is no risk of disturbance on the existing lock and its current equipment.

Management of the risk of displacements related to the realization of work

In addition of specific monitoring of the injection pressures of tie rods, displacements measurements will be made on the existing lock chamber (gate and walls) continuously during the work to manage the risk of displacement. The threshold alert will be set at 8 mm and a critical threshold at 10 mm.

Management of the risk of displacement of the bottom of excavation

There is a risk of desiccation of clays from the bottom of the excavation which may later cause the structure to move. To limit the phenomenon, earthworks in cofferdams will be completed under water and covered with concrete plug – also cast under water - before dewatering.

4.5. The existing lock

Presentation of the problem

The principle is to build the extension of the lock chamber independently of the existing lock. In addition, the downstream head of the existing lock is relatively massive and should not be affected by work downstream. However, the immediate proximity of the chamber of the existing lock requires special attention during the works.

Indeed, the latest work done near the lock (construction of the dam) showed that it could undergo displacements which, at the time, were related to the injection of tie rods under the chamber of the lock. This caused the displacement of several blocks (longitudinal displacements of 1.5 to 2 cm and vertical displacements up to 2.5 cm). At the time, they did not induce structural disorders on the lock nor on these equipment. In addition, the vibrations induced by the driving operations require special attention because of the presence of sensitive devices on the lock in operation (sensors) and risks of resonances (jacks, gate and gateway).

Management of this problem in the works phase

Before the start of the work, a test will be carried out audit the equipment likely to be impacted by the vibrations. Vibrations will be monitored during this test before threshing. If this test is conclusive, the vibration monitoring of the lock during the work will not be necessary. In the opposite case, further investigations will have to be carried out and conservative measures taken (i.e. equipment instrumentation during sheet piles and piles driving, with definitions of alert thresholds and critical thresholds).

In addition, as written above, the old lock will be followed topographically during the works to ensure that the differential movements between the different blocks do not surpass 1 cm.

5. CONCLUSION

Because of its short length, the Quesnoy-sur-Deûle lock is a singularly problematic point for the development of river traffic on the large-gauge network of the Hauts-de-France region. The choice of the building an extension using traditional reinforced concrete having been made, the technique sliding walls will be implemented to minimize closure during construction. The study made it possible to review and provide answers to a certain number of problems which will be generated by this technique, particularly in terms of cofferdam construction phasing, falsework foundation, work in Flanders clays, protection of the existing lock and, above all, respect of the deadlines.

The start of on-site works is scheduled in 2020. The implementation of this project will certainly allow to gain experience for other projects of the same type as there are many other locks to extend on the large-scale network of the region.

Miter Gate Machinery, Design Options and State of the Art

Brenden F. McKinley, P.E.

This paper is adapted from information originally presented in the report of PIANC Working Group 138.

Miter gates are used extensively on navigation locks. They usually can only be operated with equal or nearly equal head (water levels) on the gate. So, the forces that the machinery must overcome include the force needed to accelerate the gate, water resistance as the gate moves, frictional forces in the gate and machinery, and gate resistance due to small differential heads.

Linkages

Four different types of linkages are commonly used to driver miter gates. These are referred to as the Panama Canal Linkage, Fig. 1, the Ohio River Linkage, Fig. 2, the Modified Ohio River Linkage, Fig. 3, and the Direct Connected Miter Gate Cylinder, Fig. 4.

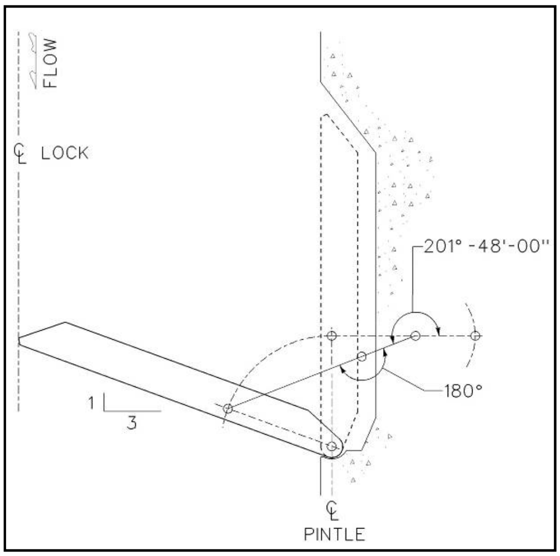


Fig. 1: Panama Canal Linkage

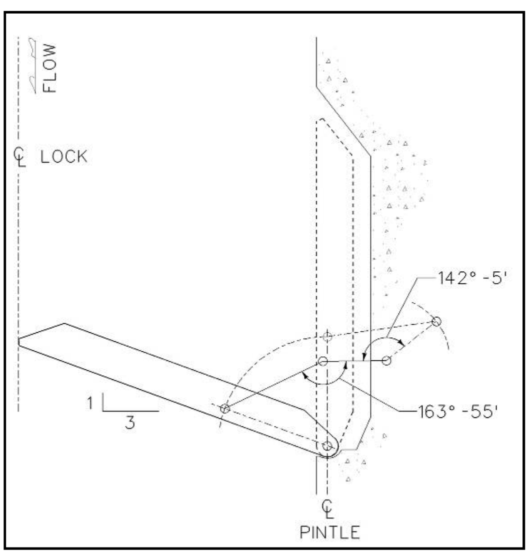


Fig. 2: Ohio River Linkage

[1] U.S. Army Corps of Engineers, Huntington District, Brenden.F.McKinley@usace.army.mil

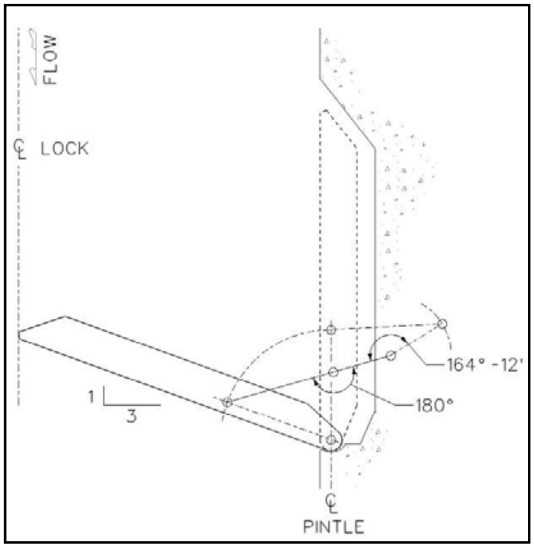


Fig. 3: Modified Ohio River Linkage

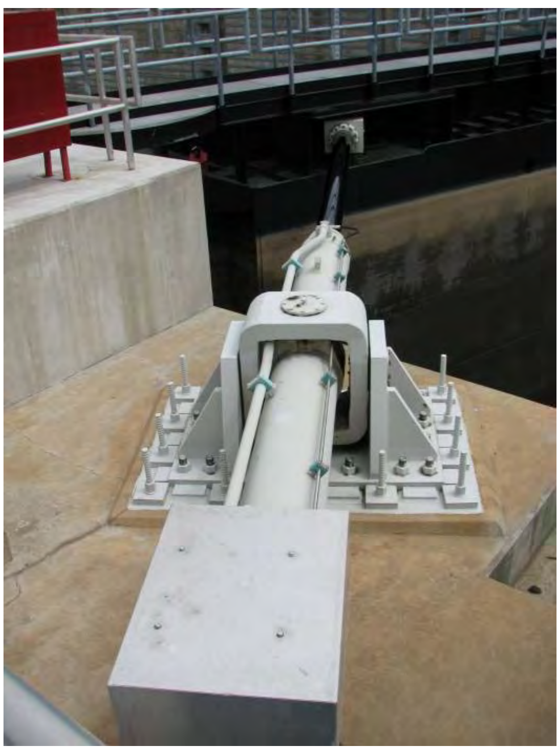


Fig. 4: Direct Connected Miter Gate Cylinder

The direct connected cylinder is used primarily on hydraulic drives. Three of the linkages, the Panama Canal, the Ohio River, and the Modified Ohio River, consist of a sector gear and arm with a strut connecting the sector arm to the gate.

The sector gear can be driven by an electric motor with gear reduction or by a rack attached to a hydraulic cylinder (see Figure 5).The Panama Canal Linkage has no angularity between the strut and sector arm in both the recessed and mitered positions. This linkage allows for uniform acceleration from miter or recess to the midpoint of travel and then uniform deceleration to miter or recess. The Panama Canal linkage also has an eccentric connection between the sector gear arm and the gate strut. The gate strut has to pass over the top of the sector gear.

Design guidance for these linkages is available from the United States Army Corps of Engineers Manual EM 2610.

The Modified Ohio River Linkage is similar to the Panama Canal Linkage. The linkage has angularity at the recessed position. The strut and sector gear, however, are located in the same elevation. This geometry eliminates the need for the eccentric strut and sector connection of the Panama Canal Linkage.

The Direct Connected linkage consists of a hydraulic cylinder with its shell (or body) supported in the miter gate machinery recess by a trunnion and cardanic ring assembly (or gimbal) and its rod connected directly to the miter gate with a spherical bearing type clevis.

The Ohio River Linkage has angularity between the strut and the sector arm in both the miter and recess positions. In the United States, the Ohio River linkage also utilizes a rack to drive the sector gear as shown in Figure 5. This system is used at locks that are submerged for months at a time and submerged on a yearly basis. The gears and rack can be quickly cleaned after being submerged. The hydraulic system is sealed even under submerged conditions.

Fig. 5: Ohio River Linkage with rack gear and sector gear

The Panama Canal, Modified Ohio River Linkage, and the Ohio River Linkage inherently decelerate as the gate moves to the miter and recess positions. This can be important if using a constant speed electro-mechanical drive. For the direct connected hydraulic cylinder drive, this deceleration should be provided for with automatic throttling of the oil flow.

While the Panama Canal and Modified Ohio River linkages offer greater deceleration at the miter and/or recess positions, greater forces can be induced in the linkage at these positions. Adjustments and compensation for temperature effects are more important with these two linkages.

The design of the Modified Ohio linkage at the miter position is critical since the linkage cannot be extended any further. End of travel adjustment is minimal and can usually only be accomplished by slight adjustments in the spring strut.

Electomechanical Drives on Panama, Modified Ohio River and Ohio River Linkages

Electromechanically driven miter gate drives in the United States and Europe have traditionally included an electric motor, gear reducer, sector gear, and a linkage which connects to the gate.

Some disadvantages of mechanical drives (as compared to hydraulic drives) include more complex operating machinery linkage and more pivot points for wear including greasing requirements (anchorage, etc). The drive is labor intensive for routine maintenance and for replacement. Components can be difficult to replace and remove, alignment can be critical, and if not done properly, the life of the machinery is shortened. The mechanical drives are generally more susceptible to shock load and barge impact (although barge impact is also an issue for direct connected hydraulic cylinders).

One of the advantages of mechanical drives (whether it is a miter gate drive, sector gate drive, culvert valve drive, etc.) over hydraulic drives is the proven design and reliability. These systems have been in place since the 1920's and 1930's in the United States and were originally installed on the Panama Canal. There is a reduced potential for significant oil contamination of the waterway at least in terms of volume. A hydraulic fluid line break or cylinder leak could result in a large quantity of hydraulic fluid being spilled into the waterway. This can be mitigated, however, by using environmentally friendly oil.

Mechanical drives, however, still have the potential for grease and oil contamination in the waterway. This is particularly the case for the open gearing, springs, and bearings. The lubricants used are transferred into the river by surge water and overflow, and consequently can cause a high degree of pollution to the waterway, which is generally unacceptable according to present day rules and regulations in many countries. Gearboxes used on mechanical drives should be raised to extent possible above flood levels or else sealed to minimize leakage.

Also, with mechanical drives, there is no longer sufficient availability assured for spare parts, such as gears. Operating components are generally custom built with long replacement lead time (ie gear box and open gears). These must be returned to the manufacturer periodically, in Europe every 10 to 50 years, for maintenance. There are limited sources for gear boxes, sector gears, screw rods, etc. and if these drives are damaged they must be repaired by the manufacturer.

When using an electro-mechanical miter gate drive, a method to dampen shock loading such as a strut spring should be incorporated to absorb forces from wave action, sudden starts or stops, or minor vessel collisions. This may be more important with an electric drive than hydraulic as the hydraulic drive can be designed with inherent shock absorption.

Use multi-speed motors or variable volume hydraulic pumps to allow a reduced (slower) gate speed at recessed and mitered position and to closely simulate the velocity profile or the Panama Canal or Ohio River linkage. The total gate operating time from recessed to miter or vice versa will depend on the gate size. For smaller gates, 25.6 meters (84-ft lock) an approximate time of 90 sec should be used and for the larger gates 33.5 meters (110-ft locks) an approximate time of 120 sec should be used.

Miter gates can usually carry water heads on one side only. In some delta areas, e.g. in the Netherlands, high water can appear on any of the two sides. In such cases, the drives or separate mechanical locking systems can be used to prevent the gate from opening by reverse water heads, see Figure 6.

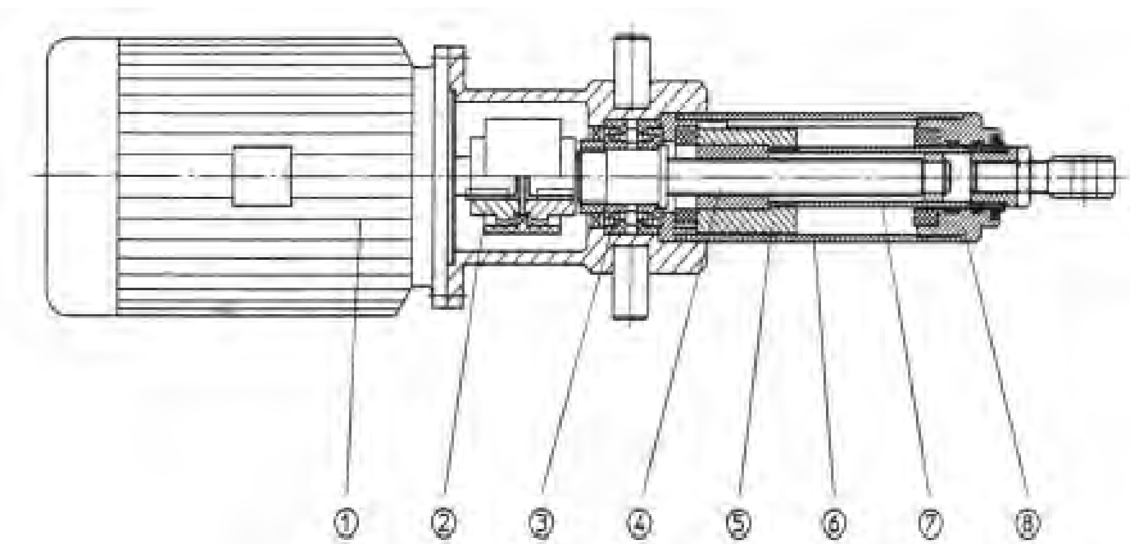
Fig. 6: Merchants' Lock in Den Helder, Netherland

Mechanical Linear Actuators

The linear mechanical electric actuator (Figure 7) is a completely encapsulated, electro-mechanical drive unit, consisting of a drive motor, screw drive with bearings, housing, and the extendable and retractable piston rod as the external power transmission element.

Mechanical linear actuators are an advancement of the mechanical drive systems traditionally used on navigation structures. The drive system is shown in more detail in Figure 8 below. The motor is designed to operate in two directions which then retracts the piston or extends the piston. The piston assembly connects to the gate similar to a direct connected hydraulic drive. A spring assembly protects the drive from minor gate impacts.

Fig. 7: Mechanical Linear Drive at a Miter Gate



1	Motor / Gearbox:	Induces torque at screw drive. Depending on the direction of rotation, it causes the actuator to travel forwards or backwards.
2	Spring assembly:	Integrated spring assembly, protects actuator against operational axial impact loads.
3	Bearing	
4	Screw drive /piston	The screw drive causes the actual move and generates axial force at torque.
5	Housing:	Its task is a general protection against external influences. The housing is used for guiding the piston.
6	Piston rod:	The piston is connected to the piston rod, which itself moves inwards and outwards.
7	Guiding of piston rod:	Its task is to guide the piston rod and at the same time it is a sealing element, also protecting against dirt and aggressive medium.
8	Swivel- / fork head:	Trough the pivot head / fork head, the power is fed to the part to be moved.

Fig. 8: Linear Electromechanical Actuator

With the mechanical linear actuator, forces to approximately 1,000 kN and a maximum (extension) of 5,300 mm can be transferred. The fundamental construction consists of an electric motor which actuates against the system. A screw drive brings this force linearly on the piston rod. Depending on rotation direction of the electrical motor, an ahead or back movement of the piston rod is realized.

The advantages of the linear mechanical actuator, compared with conventional mechanical drives, are documented by several references in the area of hydraulic steel structures. In general, the linear mechanical actuator will be simpler in construction and available as a standard item from multiple manufacturers. Lubrication and maintenance are reduced. It will eliminate the need to custom design and build a drive system.

The linear mechanical actuator is similar to the self-contained hydraulic actuator discussed later. The primary advantage of both types of units is that they are completely self-contained. This would also be the primary advantage of using the linear mechanical actuator over a typical hydraulic drive with a direct connected cylinder.

The totally enclosed, electric linear actuator, with specially selected materials of construction, long-life internal mounted screw drive and gear motor drive, has many fundamental advantages including:

- Lubricants cannot leak into the water due to the fully enclosed design and high-grade seals employed.
- The linear push-pull forces are generated electro-mechanically as opposed to a hydraulic drive system.
- Integrated adjustable controls provide for stroke adjustment and control of maximum thrust force to provide precise control of gate motion and firm closure.
- Maintenance costs are reduced substantially due to high wearing life of components and long interval lubrication.
- Standardized construction minimizes spare part requirements
- High mechanical efficiency drive system combined with power used only when operating means lower energy costs
- Low noise emissions <60dB (A) possible

Although the linear mechanical actuator can produce fairly significant torque, it is still limited when compared to a traditional hydraulic direct connected cylinder design or a traditional mechanical drive system.

The mechanical linear drive should contain an integrated package of disc springs to avoid hard axial impacts. The spindle system should be designed as a trapezoid thread or ball thread or a controlled planet role thread. The completely corrosion-protected piston rod fence is to be built against IEC Ingress Protection (IP) 68 as well as with dirt and ice wipers. The speed and torque supervision occurs with a frequency converter. An emergency hand service is made by through a hand wheel.

Stress analysis of load-transferring components is performed according to European Standard DIN19704.

Linear Hydraulic Actuators

Linear hydraulic actuators or cylinders are used throughout international waterways, including miter gates, various valve types, arresting mechanisms and other associated equipment. Cylinders may vary in size and construction depending on the application. Cylinders can come in different construction, tie rod, screwed, bolted, welded or a combination. The waterside applications dictate the need for a robust construction with specific attention made to the rod material and coating method and rod sealing systems. Cylinder heads may be of:

- Detachable cylinder end (bolted); the removable cylinder base is bolted to the cylinder.
- Butt welded cylinder end; no bending moments transferred.

For maximum fatigue strength, the second option is recommended.

Fig. 9: Large Cylinder operating a miter gate

Cylinders allow a link directly to the driven component avoiding intermediate linkages and all that is associated with them; including transmission losses, alignment, and additional maintenance. Hydraulic cylinders, provide high forces in a compact size, as well as reduced down time of the equipment replacement in the event of failure.

Location and hydraulic supply lines feeding the hydraulic cylinders can be challenging as is the potential of oil discharge from rod seals, resulting in contamination of the waterway.

Larger bore and stroke cylinders require consideration for handling, this may include special attachment points and adapters to facilitate maintenance and installing/removing.

All spare cylinders (including those awaiting to be installed) should be fully retracted and mounted vertically. If this is not possible, then the cylinder should be rotated by 90 degrees periodically. This is necessary so that the piston and rod seals do not generate flat spots due to the excessive weight of the cylinder rod and piston assembly. Otherwise, this will result in under-performing seals when put in service. It is also preferable to ensure piston seals are energized under pressure on both sides while not in use.

When designing a system, minimizing spare cylinders requires consolidating drive cylinders into a few sizes to match all applications across an installation. Careful attention should be given for proper design of force loading, especially when retrofitting existing mechanical installations.

Heavy Duty Mill type cylinders rated at 25 MPa (3625psi) design pressure, bolted/screwed construction will provide an extended life of operation when maximum system pressures are limited to 85% of the design pressure.

Low pressure cylinders (< 16 MPa) are not recommended. The reason is that the lighter cylinder walls are not suitable for corrosive environments.

Quality protective finish is strongly recommended because the high initial expenditure cost will be returned over time.

Base plates for connecting pipes and valves should be mounted directly on the cylinder prior to finish painting.

Pipe thread connection of cylinders should be considered only for cylinders with an inner diameter smaller than 180 mm and a maximum pressure less than 10 MPa.

For cylinders that will be extended for long periods of time, special attention should be paid to cleaning the cylinder rod. This includes providing a hard wiper seal at the rod end entry into the cylinder. Secondary wipers accommodating external discharge of material from the inner wiper seal should also be included.

Miter gate cylinders should have extended support to the rod, by providing high load bearings on the rod and piston seal areas.

The use of rod protective bellows, are to be avoided and not recommended. The bellows become split and trap material inside between the rod and the inside of the bellows. Over time, the trapped materials will damage the rod seals.

Compact Hydraulic Power Packs

A technology that has proven successful in German installations is a Compact Hydraulic Drive (CHD), which is a compact pumping system that can be placed near each component, generally a cylinder.

All compact drives, whether indoor or outdoor, are made as environmentally isolated systems in order to minimize the effects of humidity and contamination of fluid. No fluid changes are required during the life of the unit. Outdoor CHD units are equipped with an overpressure tank, where the fluid level can expand in the existing free space by compression of the air. This is done to absorb the pendulum volume of the hydraulic system and also the volume change due to temperature changes.

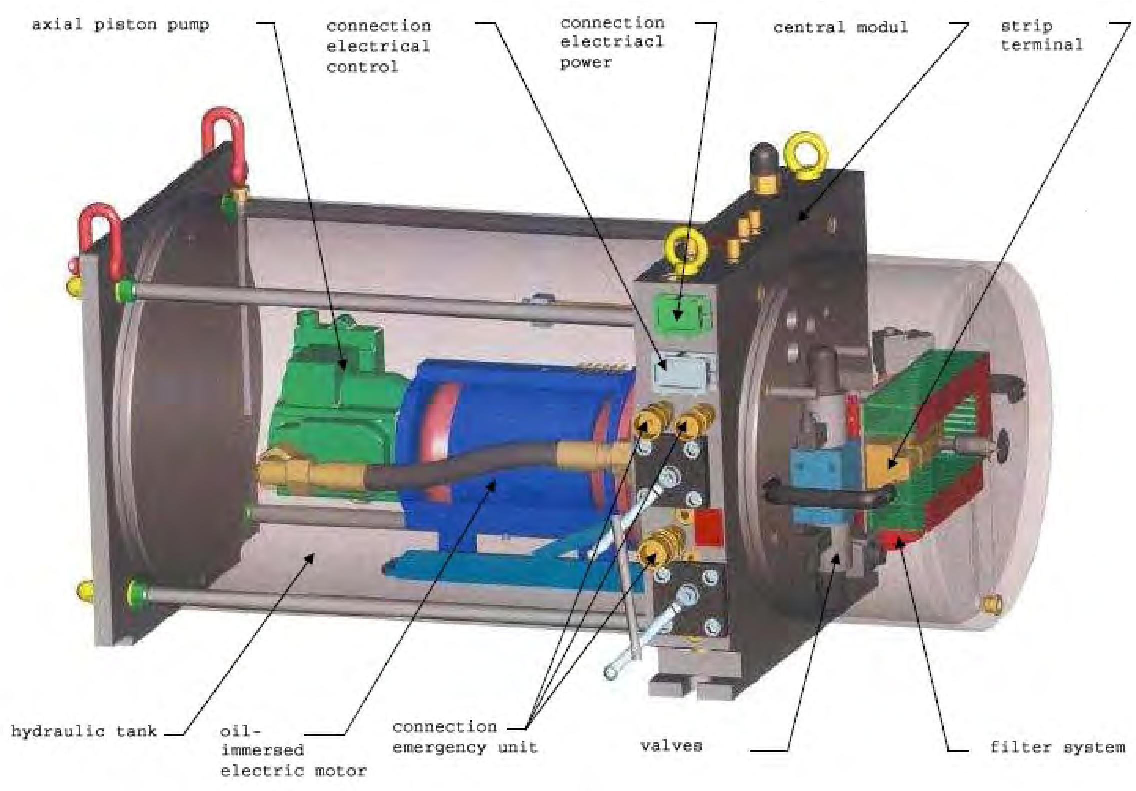


Fig. 10: Hydraulic Compact Drive unit - general arrangement

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The hydraulic compact units are driven by variable speed motors for adapting the speed of cylinders to the respective operational specifications. With variable speed motors, robust constant flow pumps and simple on/off valve techniques for compact units can be applied. In addition to the position sensing of the piston rod, further process parameters such as operation pressure, oil temperature, oil level and filter contamination are monitored by remote control so that disturbances and faults can be detected.

Piping/hosing is designed with hydraulic quick-release couplings, and the cabling is designed with plug contacts of enclosure type IP 68, which are safe against flooding. This permits quick and uncomplicated replacement of units within approximately. 10 minutes. Environmental grade fluid used in the unit is synthetic ester (water hazard class 0).

The hydraulic compact unit is composed of a drive module, a valve module and a connecting central module.

Fig. 11: Typical Hydraulic Compact Unit

The drive module serves as a tank for the hydraulic fluid as well as a drive unit. A pump combination, composed of an axial piston pump and a flange-mounted oil-immersed electric motor are arranged in the drive module.

The drive module operates without maintenance, and the fluid level is automatically checked. The drive module is connected to a central module. The central module serves simultaneously as an anchoring unit for the drive module as well as for all valves.

The central module is also the receiving unit for all electrical cables and plugs, all hydraulic power, control and emergency operation connections, and all control valves. Because of this arrangement, there is no requirement for piping since all hydraulic connections are provided by borings inside the central module. The valve module, also designed as a round vessel and mounted at the opposite side of the central module, is oil free and is composed of a pressure filter with contamination control, the two way-valves of the operation cylinder, and a temperature limiting switch. The interior of the valve module is designed in such a manner that a complete control module can be integrated. This means that the usual control boxes outside the drive are not required.

Electric plug connections (IP 68 rated) are mounted on the central module. Power and control cables, which run from an external control cabinet, are placed into these plug connections.

In the case of damage, quick disconnection can be achieved by releasing the plugs. The compact drives are constructed in weather-resistant aluminum in a water-proof, flood safe design. All outgoing hydraulic and electrical connections are made of stainless steel.

As with the compact units, the drive cylinders are connected by means of hydraulic quick-release couplings and electric plug contacts. These electric plug contacts are position sensing, and have a 24 V valve supply for emergency lowering of locking elements.

The mechanical interconnections of cylinders are designed in such a manner that they can be quickly removed from the maintenance-free bearings with a minimum of effort. Cylinder-specific functions, such as rapid extension at the gate and emergency lowering of the locking elements, in case of power failure of the cylinders, are taken over by the control block mounted directly on the cylinder. Because the cylinders each have their own control blocks with pressure limiting and load holding valves, adjustments at the compact unit are unnecessary. As a continuation of a maintenance-friendly drive concept, the magnetostrictive internal position sensing systems of both the gate and locking cylinders are equipped with a pressure tube, which allows replacement of the sensor without draining of pressure fluid.

Service-Board and Emergency Unit

A further important aspect of the design is the reliability concept of the drive. With the aid of the mobile service-board, it is possible to operate and monitor a compact unit during maintenance and overhauling work in the repair facility, as well as during complete failure of the lock control system. When necessary, simply decoupling the electric plug connectors will allow a controlled manual operation to be carried out.

Operation of the cylinders is also possible with the aid of the mobile emergency units. These can be easily connected to the respective compact unit by means of quick-release couplings without further reconstruction activities. Switching from normal to emergency operation is performed simply by pushing a ball valve. The emergency unit is composed of a basic frame on which is arranged a small gear pump–electric motor combination, manometer, manual valve with pressure limiting valve, as well as three hoses with attached quick-release couplings. The emergency unit can also be operated by means of a small emergency power generator.

All compact drives whether indoor or outdoor are made as environmentally isolated systems, so that neither dampness nor pollution can damage the oil. This kind of construction guarantees the whole life-cycle without oil changes, as the oil is protected against water and dirt entry and the oil properties will not degenerate.

To take the pendulum volume of the hydraulic system and also the volume change rate by temperature changes, the tank of the outdoor unit is constructed as an overpressure tank. Therefore, the liquid difference amount can be accommodated in the tank by compression of the air in it. With the indoor system, an additional outside and maintenance-free hydraulic compensator is used as an adaptable membrane or reservoir separator bag.

Self-Contained Hydraulic System

A self-contained hydraulic drive system combines a hydraulic power unit with a hydraulic cylinder to form a self-contained actuator that is completely sealed and submersible.

Instead of directional valves, the drive unit utilizes a bi-rotational gear pump mounted inside a sealed reservoir that is driven by a submersible and reversible electric motor. The speed and direction of each actuator is controlled by a variable frequency drive (VFD) which controls the speed and direction of the electric motor.

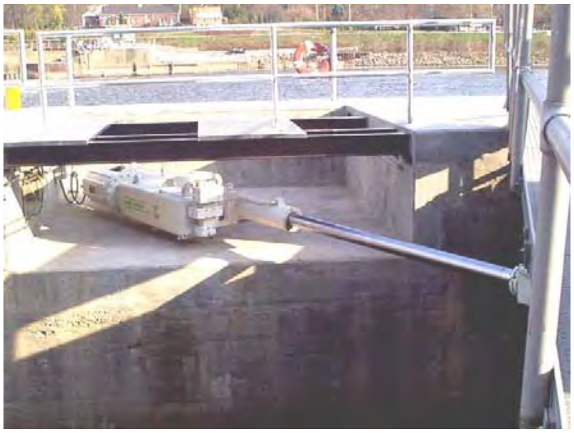

Fig. 12: Self-Contained Drive System – United States Army Corps of Engineers Pittsburgh District

The gear pump is a simple, rugged, positive displacement design. Gear pumps have a high tolerance for fluid contamination, good overall efficiency, and are relatively quiet. While these pumps provide a fixed volume at a given speed (rpm), their flow rate/speed characteristics are linear within their efficiency range. Speed and direction control of an actuator can therefore be provided by driving a reversible gear pump with a variable speed electric motor, which makes them ideal for self-contained type power units. Gear pumps are commonly used for pilot pressure applications. Gear pumps are generally restricted to less than 24 MPa (3,500psi) service.

Sealed reservoirs are primarily used for the power unit of a self-contained actuator. This consists of a power unit attached directly to the hydraulic cylinder it operates. These actuators can be configured in many different ways by changing the design (shape) of the reservoir and where and how it is attached to the cylinder. The direct connected miter gate actuators recently installed on several locks have long slender reservoirs, made from square structural tubing, bolted to brackets on the side of the cylinder.

Sealed reservoirs have a pump mounted inside and submersible motor mounted outside. Since these reservoirs do not have breathers or accumulators, the air pressure inside will vary with cylinder rod position and oil temperature.

The actuator should be designed so the normal pressure range in the reservoir is between 21 kPa and 69 kPa (3 and10 psig). Care should be taken to make sure the pressure never goes below atmospheric pressure or above 207 kPa (30 psig).

Advantages

- Completely self contained – no external piping or motors or hydraulic power units – sealed from dirt and moisture
- Piping friction losses are eliminated
- Reduced total space requirements
- Low maintenance
- Easily replaceable – plug and play
- Force and speed fully adjustable for each direction of travel
- Smooth vibration free operation
- Weather proof and can be provided as an explosion proof unit. Can be installed in various mounting configurations.
- Submersible

Disadvantages

- Limits to cylinder size

- A back-up unit needs to be stored on-site for replacement

Strut Springs

Spring type miter gate struts are commonly used with the Panama, Ohio River, and Modified Ohio River type linkages (Figures 1, 2, and 3). Springs are built into the strut assembly in order to, when compressed, act as a shock absorber to soften the loads transmitted to the operating machinery.

The spring-packages are also used to keep the miter gate leaves under a certain pre-tension in open position, to prevent them from moving under variable water loads. At least three types of springs have been used in this application. These include coil springs, ring or Belleville springs (Fig. 15), preferred in The Netherlands).

Fig. 13: Miter Gate Strut Spring

Miter gate strut springs can also act as a shock absorber for barge impact loads at least for smaller impact loads. Strut springs should also be designed for small amounts of travel adjustment of the linkage and for adjustment of tension.

Fig. 14: Miter Gate Strut Assembly

The advantage of strut springs is to decrease the transmission of shock into the machinery. This is particularly important in electro mechanical drives including miter gate linkages such as the Ohio River and Modified Ohio River.

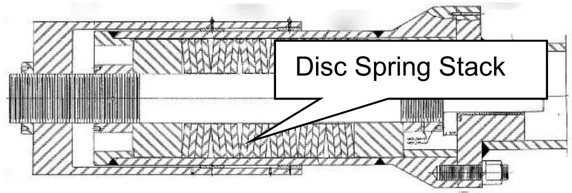

Fig. 15: Strut Spring Assembly

A disadvantage is that they require periodic maintenance and lubrication. Also, they are a common failure point in the miter gate linkage. When the spring breaks, it creates unrestrained movement or play in the strut and results in difficulties in positioning the gate either in the miter or recess position.

Coil springs take the most space but can be obtained from multiple manufacturers. Ring springs are highly efficient and have a high spring constant or a high capacity per unit of spring weight or volume. A disadvantage of ring springs is that they are available from a limited number of manufacturers.

It is important to maintain lubrication of the strut springs, particularly the ring spring and disc spring types, in order to ensure maximum life. Usually this means filling the spring chamber with grease.

If a strut spring is broken, causing play in the linkage, the exact position of the gate will be unknown unless sensed directly from the gate or from the machinery. This can be a problem if the position of the hydraulic cylinder or other component of the drive train is used to indicate gate position. This could then lead to a false indication of gate miter.

Supporting of Miter Gate Cylinders

Fig. 16, 17, and 18Fig. show the design of a cylinder support and the linkage of the cylinder to a gate, which is designed for a quick replacement of the cylinder.

Furthermore, this design is able to be used in existing drive pits at underground level, without the need for civil work. The existing steel foundations are utilized for the drive. The existing trunnions are removed and the new support structure is bolted to the steel foundation by means of a base plate.

Fig. 16: Supporting of gate cylinder for quick replacement

The cardan joint, where the drive cylinder is installed on its central axis, is designed as a closed, compact frame with bearing journal. The gate cylinder can be very quickly dismounted and replaced by simply pulling both exterior bearing journals.

Fig. 17: Cardanic support of cylinder, Bernhardsluis, Wijk bij Duurstede, Netherlands

Piston rods and gates are connected with a pin using a frictional connection. A spherical plain bearing is mounted on the piston rod end. The interior ring of this bearing is fitted with a tapered sleeve. This sleeve is pressed onto the taper of the floating gate pin by means of a bolted connection.

Fig. 18: Quick connect linkage of cylinder to a miter gate

When the hydraulic cylinder is to be separated from the gate, first the covering screen is dismounted, then the sleeve is pulled from the pin's taper by means of an extractor screw, whereby the frictional connection is released. This design eliminates the need for precision insertion of the pin and spacers.

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Selection Strategy of Failure Modes for Repair and Maintenance Activities

by

François Nyobeu[1] and Andreas Panenka[2]

ABSTRACT

Strategic decision making about repair and maintenance activities is increasingly becoming a major concern in a context where navigational assets are aging and suffering various deterioration mechanisms, affecting their structural reliability. This paper examines the effects of various failure modes on the structural reliability of navigational assets and introduces priority weights that are used to rank different failure modes with respect to the ensuring riskiness of infrastructure failure. Based on previous findings of a qualitative as well as quantitative analysis of the emerging risk of infrastructure failure by means of the Failure Mode and Effects Analysis (FMEA), a Multi-Criteria Decision Making (MCDM) Method is used in this study to take into account not only the vagueness and subjectivity of expert knowledge, but also to take into consideration uncertainties of available damage data. In a previous study the Severity (S) of the consequences of a potential failure, probability of Occurrence (O) of the failure mode and probability of Detection (D) of the cause of the failure were identified as overriding criteria for assessing the emerging risk of infrastructure failure, using the Failure Mode and Effects Analysis (FMEA). Applying a Multi-Criteria Decision Making method (MCDM), namely the Analytic Hierarchy Process (AHP) to these criteria, priorities weights are developed to enhance decision making about repair and maintenance activities. Our findings revealed four categories of failure modes that can be summarized into failure modes that may hinder the durability of the infrastructure and failure modes that are likely to alter the structural reliability of the structure in a very short term. The proposed methodology also takes into account influential aspects of various failure modes as well as the subjectivity of expert judgments. This paper contributes to shed more light on how past damages data and expert knowledge can be combined to infer key figures that are useful for the identification of failure modes for repair and maintenance actions.

Keywords: Maintenance and operation, Multi-Criteria Decision Making (MCDM), Analytic Hierarchy Process (AHP), Priority weights

1. INTRODUCTION

The sustainable economic growth, productivity, resilience and competitiveness of the German economy, in the prevailing context of repetitive economic crises as well as the well-being of the German population heavily depend on the reliability and durability of its infrastructure assets. Accordingly, German authorities, in charge of navigational assets own the responsibility of managing a network of facilities that meet its intended requirements of availability, operability, structural reliability and safety during the intended service lifetime. However, the heterogeneous portfolio of German navigational assets, including navigation locks, weirs, uplifts, sag pipes, bridges, etc. is faced with various deterioration processes, associated with aging factors and environmental stressors. Especially, the ongoing deterioration processes may impede the serviceability, the ultimate load-carrying capacity and the durability of the navigational assets. In addition to the structural

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degradation, failures of these infrastructures could pose the risk of secondary cascading effects, whose consequences may be far beyond a simple loss of service.

Age might be regarded as an important factor that considerably impacts the structural reliability and functionality of these assets, and thus, fosters their vulnerability against threats posed by changing environmental conditions, insufficient maintenance activities. At the same time, deferred maintenance activities constitute huge burdens that urgently need to be addressed. More specifically, almost 30% of the navigation locks and weirs of the Waterways and Shipping Administration (WSV) were constructed prior to 1900. Beyond merely having already exceeded their designed service lifetime of about 100 years, some of these assets, which should have already be replaced are still in use, despite numerous deterioration processes, which may have significantly altered their structural reliability. While these facilities are aging and suffering various types of deterioration mechanisms, their level of service, structural reliability are decreasing and the potential risk of infrastructure failure is increasing. Likewise, several navigational assets that were built in the early 1960s, for instance at the Main-Danube-Canal, are exhibiting unexpected deterioration processes, which significantly affect not only their serviceability, but also their structural reliability. Although the renewal process of some of these infrastructures is engaged, it is important to acknowledge that design errors, inadequate and deferred maintenance are the main causes of this degradation.

Again, owing to restricted investment budgets and insufficient maintenance personnel, reinforced concrete and hydraulic steel structures of the German navigational assets have not been properly mended for decades. Yet, infrastructure serviceability highly depends on the frequency and quality of maintenance activities. Likewise, aging materials, insufficient maintenance and excessively prolonged service lifetime contribute to weaken these infrastructures, and thus, they become more vulnerable to otherwise environmental stressors. Because of the broad diversity of assets, the German Waterways and Shipping Administration (WSV) has since 2008 established a Maintenance Management System (EMS-WSV) to support decision-making process regarding the prioritization of repair and maintenance activities. As a result of the maintenance backlog, existing assets are deteriorating much faster than necessary and the well-established EMS-WSV is progressively reaching the limits of its capability.

This paper introduces a method to enhance decision-making with respect to prioritization various failure modes, identified in a Failure Mode and Effects Analysis (FMEA), taking into account the potential riskiness of failure modes concerning functional and structural requirements of an asset. The Analytic Hierarchy Process (AHP) developed in this study offers the chance of searching and evaluating the causal relationships between various criteria and failure modes. Furthermore, the proposed AHP can help to understand the absolute riskiness of individual failure modes in an overall perspective, and thus, to establish a priority order of failure modes for maintenance. A comparative analysis of several failure modes that affect reinforced concrete components of navigation locks is described in this paper to illustrate the procedure. The rest of the paper is structured as follows: section 2 presents a brief overview of the existing Maintenance Management System (EMS-WSV) of the WSV. Section 3 primarily describes the methodology used in developing our key figures and section 4 discusses a case study based on different failure modes, affecting reinforced concrete assets, to demonstrate the practical application and effectiveness of our proposed AHP.

2. **ASSET INSPECTION/CONDITION ASSESSMENT**

2.1 Inspection procedure

Current decision-making strategies for repair and maintenance activities of navigational assets in Germany are based on analysis of damage data gathered during regular visual inspections. The definition of type of inspection, the extent and scale of execution of these inspections are characterized by a time-based approach. This is also in line with the structuration of navigational infrastructures that are clustered into two main categories: category A and category B. While assets of category A undergo Principal Inspections, Surveillance and Observation, assets of category B are just observed (BMVI, 2009). Principal inspections are carried out every 6 years with surveillance inspections at an intermediate period of 3 years. During principal inspections, the structural reliability, including the load-carrying capacity, the serviceability and durability of every single component of the asset is investigated by a certified civil engineer (BMVI, 2009). Hence, every distinct damage is gathered, evaluated and documented according to its effects on the structural requirements (structural reliability and serviceability) of the component and the asset as a whole.

The intended objective of principal inspections is to detect, evaluate ongoing deterioration processes and to make informed strategic decisions about the neediness and acuteness of repair, maintenance or potential replacement. Indeed, principal inspections are performed to determine the current condition of assets and identify prevailing structural deficiencies. For example, if evidence exists during an extensive inspection that a particular damage (crack, corrosion of the reinforcements, etc.) may impair the reliability/functionality of an asset, actions must be taken to prevent a complete failure of the component, and thereafter, avoiding a potential collapse of the asset.

During these in depth inspections, all gathered damages are rated on a scale from 1 to 4 (1 being good condition (CG_{gen}) and 4 being bad or "critical" condition). Using the Maintenance Management System "WSVPruf", the overall condition grade of each infrastructure is generated, based on all collected damages. The decision-making about which component/asset should be maintained is largely governed by the overall condition grade of such components/assets.

2.2 Results of the current condition assessment procedure

Figure 1 compares the overall condition grade of navigation locks and weirs with the condition grade of various reinforced concrete components of these facilities.

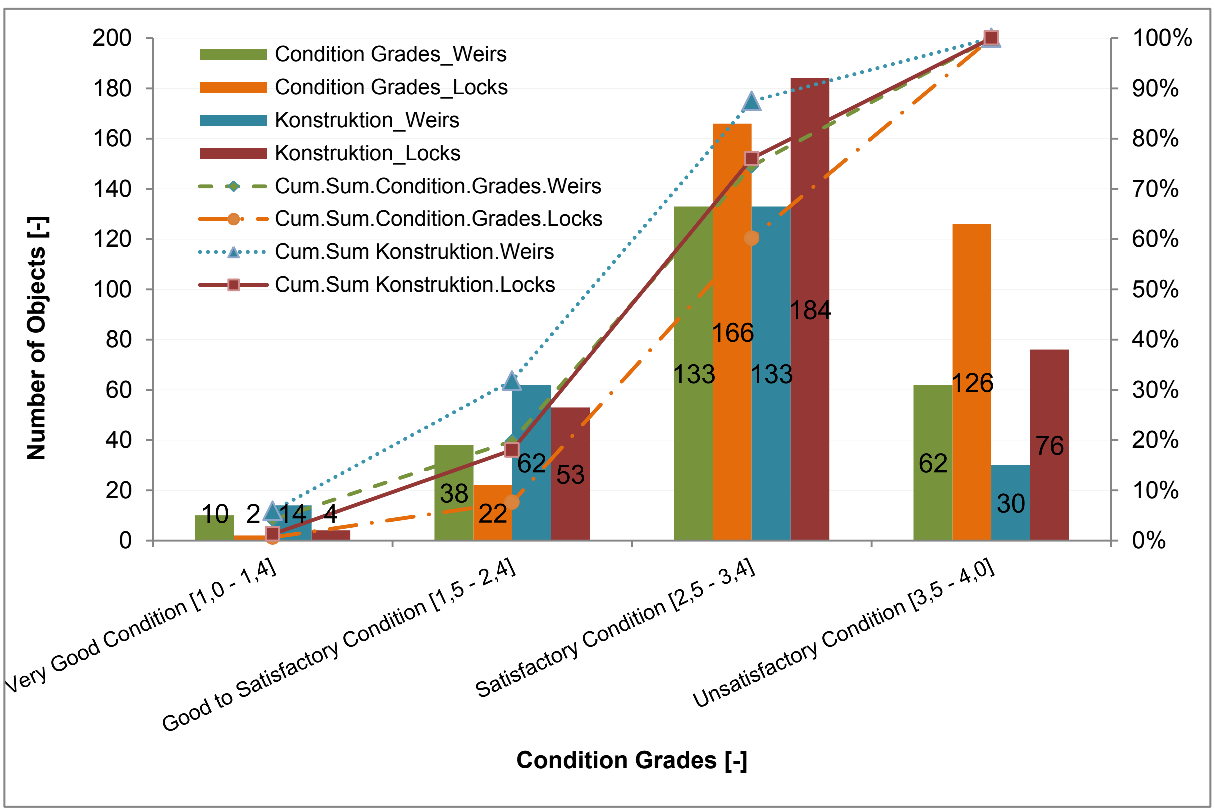

Figure 1: Condition grades distribution of whole assets and reinforced concrete structure "Konstruktion" for navigation locks and weirs in Germany (WSVPruf, 2018).

It is apparent that the bulk of navigational assets (locks and weirs) are currently rated as having a either a satisfactory or an unsatisfactory condition. This in turn means that the load-carrying capacity, the serviceability and durability of these facilities may be threatened and that significant maintenance actions might be needed in a near future. More precisely, 33.54% and 12.55% of navigation locks and weirs respectively are already in a “critical” condition and suffer severe deterioration processes. This group of assets represents the current backlogged maintenance of the WSV that urgently needs to be addressed. In enhancing decision-making about infrastructure maintenance, the Federal Waterways Engineering and Research Institute (BAW) have since 2008 initiated within the framework of the EMS-WSV a forecasting model. Based on current condition grades, the BAW uses their forecasting model to predict the future condition of an asset. The main objective if the forecasting model is to estimate the probable remaining service lifetime of various aging infrastructures and to identify different assets, which owing to their actual condition, pose an eventual risk of infrastructure failure in a relatively near future.

2.3 Shortcomings of the current condition assessment approach

While the current overall condition grade merely expresses the neediness of undertaking repair/renewal measures, it does not explicitly take into account structural effects of damages on the structural reliability of the asset. Thus, a further differentiation between the large number of assets presently rated with a critical condition grade of 3.8 to 4 is not yet possible. Furthermore, uncertainties and subjectivity inherent to inspection procedures based on regular visual inspections as well as expert knowledge are not yet straightforward addressed in the decision-making procedure. Last but not least, the current condition grading system is in reality governed by a single worst damage, detected on the structure or component. Therefore, the influential and combined effects of a considerable amount of gathered damage data is currently not incorporated in the overall condition grade. Thus, their potential effects of the overall structural reliability of the infrastructure are not considered and their potential riskiness is simply ignored. Under these circumstances, the transparency, comprehensiveness and conclusiveness of the current decision-making process regarding repair and maintenance activities should be questioned and the emerging risk of infrastructure failure should also be integrated in the current condition grade.

3. DECISION-MAKING BASED ON QUALITATIVE RISK ANALYSIS

The growing technical complexity of large engineering systems and industrial products, together with the increasing public concern over their safety has stimulated the research and development of novel risk analysis and safety assessment procedures (Bhushan and Rai, 2004). However, modelling and analyzing complex risk scenarios increasingly require historical damage data as well as profound expert knowledge that are sufficiently expressive to be qualitatively as well as quantitatively analyzed. Besides, in risk assessment of large engineering systems, it may be difficult to employ probabilistic risk assessment approach in situations where data are mainly available in form of expert knowledge or qualitative information.

3.1 Making use of expert knowledge - Failure Mode and Effects Analysis (FMEA)

Initially developed in 1960s as formal design methodology in the aerospace industry for ensuring the structural reliability and safety requirements, the FMEA has been extensively adapted in various sectors, including automotive, nuclear, electronics, chemical and medical technologies (Gilchrist, 1993; Bertolini et al., 2004; Bowles, 2004; Liu, 2016). However, to the state of our knowledge, the method has not yet been explicitly employed to assess the structural reliability of civil engineering infrastructures. Likewise, various definitions of the FMEA, depending on pursued objectives, types of FMEA and field of implementation are available in the abundant literature. According to Stamatis (2003), the FMEA can be defined as “*an engineering technique used to define,* *identify and eliminate known and/or potential failures, problems, errors and so on from the system, design, process, and/or service be-fore they reach the consumer*." Similarly, Lui (2016) describes the FMEA as "*a systematic methodology designed to identify known and potential failure modes and their causes, and the effects of failure on the system or end users, to assess the risk associated with the identified failure modes and prioritize them for proactive interventions, and to carry out corrective actions for the most serious issues to enhance the reliability and safety of products and processes, design or services*".

In essence, the FMEA heavily relies on expert knowledge, and thus, is considered as a qualitative risk assessment approach. The FMEA is supported by cause and effect chains, which are used to identify potential design and process failures of a system prior to their occurrence. In doing so, emerging risk of failures is minimized by either proposing design changes or by adjusting operational procedures whenever the aforementioned designed modifications cannot be formulated. As Lui (2016) pointed out, the main objective of the FMEA is to assist analysts in prioritizing failure modes of a system, design, process, product or service in order to adequately assign the limited resources to the highest risk items. Although a traditional FMEA can be summed up in a two-step procedure, namely the analysis of the system (Steps 1 to 3) and the risk assessment or criticality assessment (Step 4 & 5), the FMEA-implementation includes the following main steps:

- Step 1: delineation of the structure and components of the system to be analyzed;
- Step 2: definition of functions of the selected component, subsystem or system;
- Step 3: identification of failure modes and failure causes for each component as well as the effects of failure modes on components, subsystems and the entire system;
- Step 4: computation of the Risk Priority Number (RPN); depending on the RPN, rank and identify failure modes that urgently need to be maintained;
- Step 5: suggest improvement measures to mitigate the risk and enhance the system performance.

The definition of failure modes, failure causes and failure effects in a conventional FMEA depends on the level of analysis and the criteria used for the assessment of the risk of failure. Thus, the FMEA is regarded as a hierarchical modelling process aiming at constructing an attribute tree, which can either be achieved by using a top-down or a bottom-up approach. Accordingly, an essential task in analyzing a system or infrastructure asset is the hierarchically decomposition into its main components (see Panenka & Nyobeu, 2018b). One of the main ideas behind the structural decomposition of an asset into its main constituents is the necessity of identifying relevant components that significantly contribute to the fulfillment of structural and performance requirements. Also, during the hierarchical structuration of the asset, sub-functions as well as main functions of different components of an asset are identified and interfaces between various components are well established. Thus, having identified the essential components of an infrastructure, one can employ the FMEA to identify, evaluate and mitigate potential failure modes that may hinder the structural reliability of these components.

The FMEA provides both qualitative and quantitative measures to identify failures modes and assess their effects towards the quality/structural reliability of products/systems. In a conventional FMEA, the risk priorities of failure modes are determined through the Risk Priority Number (RPN), which is the product of the probability of occurrence or the frequency of a failure mode (O), the probability of detecting a failure before the realization of it impact (D) and the Severity or seriousness of the effects of the failure mode (S) (see Figure 2).

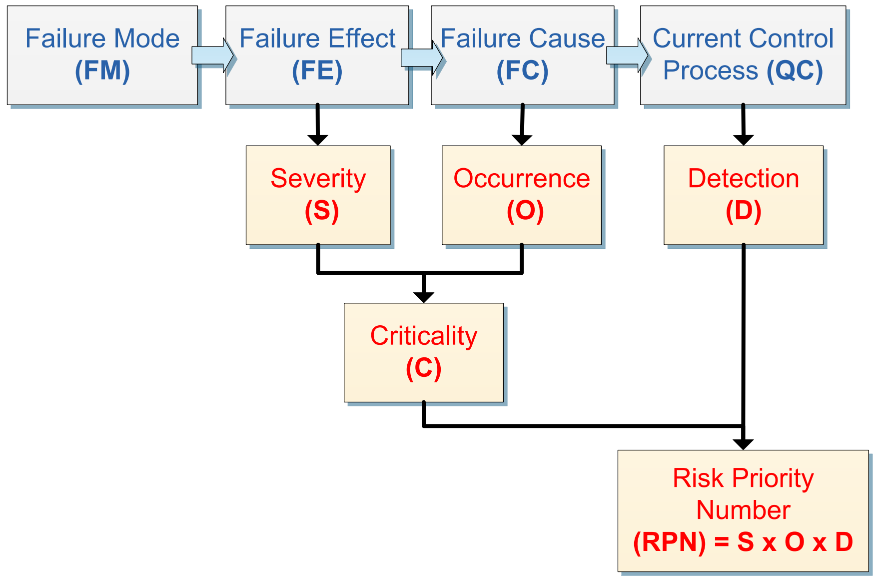

Figure 2: Cause-Effect Chain in a traditional FMEA

It is also important to stress that additional relevant risk criteria, including economical aspects, safety, equipment/system importance, maintenance cost, failure rate, mean repair time and operation condition are not often considered in analyzing the risk of failure. Thus, the RPN is computed by assigning to the three selected risk criteria values ranging from 1 to 10. This results in RPN ranging from 1 to 1000, with higher RPNs being assumed to pose a greater risk of structural failure than lower RPNs (Lui, 2016).

3.2 Enhancing the risk prioritization through Analytical Hierarchy Process (AHP)

Although the FMEA may capture the decision maker´s intuition, judgment and experience, one of the main shortcomings is its indifference to the subjective impression that one failure mode (FM) or risk factor may be of great importance for the risk assessment than others. This subjective judgment is not yet sufficiently taken into account by the RPN, computed in the traditional way by merely multiplying the values of the three risk factors O, S and D. To strengthen the risk assessment procedure and enhance the capability of the conventional FMEA by considering these potential relationships among various causes of failure or risk criteria (i.e. O, S and D), the Analytical Hierarchy Process (AHP), initially developed by Saaty in 1980 (Saaty, 2008) is used in this study. Instead of assessing each FM by separately determining the RPN, the AHP is used to determine the influential weights of each Failure Mode, and thus, various failure modes can be prioritized accordingly.

The AHP is a powerful and flexible multi-criteria decision-making tool that hierarchically decomposes a challenging decision problem into different levels of decision, where both qualitative and quantitative aspects are considered. It also combines both subjective and objective judgments into an integrative framework based on ratio scales from simple pairwise comparisons and assist the analyst to organize the critical aspects of a problem into a hierarchical structure (Saaty, 1990 & 2008). Similarly, Saaty (1987) describes the AHP as "*a nonlinear framework for carrying out both deductive and inductive thinking without use of the syllogism by taking several factors into consideration simultaneously and allowing for dependence and for feedback, and making numerical trade-offs to arrive at a synthesis or conclusion*." Because significant failures are usually caused by combinations of two or more failure modes, the AHP is employed to determine the influential weights of various failure modes and prioritize the failure modes accordingly. While the AHP employs a unidirectional hierarchical relationship among decision levels, interdependence among various components of the system is also considered and composite priority weights are generated through the development of a “super-matrix” (Saaty, 2008). The AHP is a three step process, which hierarchically breaks down a complex problem. Within this hierarchical structure, the overall decision objective lies at the top of the hierarchical inverse tree, various criteria, sub-criteria are arranged on the next level below the main goal and decision alternatives are displayed at the tail of the hierarchy. Decision makers then compare each factor to all other factors at the same level of hierarchy using a pairwise comparison matrix to find their local priority weights or relative importance. The optimal solution is the alternative with the greatest cumulative weight (Saaty, 1990). It is however essential to note that the adjective “relative” is used in this context because the obtained criteria priorities are evaluated with respect to each other.

3.3 The proposed FMEA-AHP methodology

Aiming at enhancing the expressiveness of the current condition grade within the EMS-WSV, the proposed methodology is used not only to establish influential relationship between structural system requirements and possible risks of a system failure, but also to generate additional key figures, which may support the development of joint maintenance and expansion strategies (See Figure 3, Panenka and Nyobeu, 2018b).

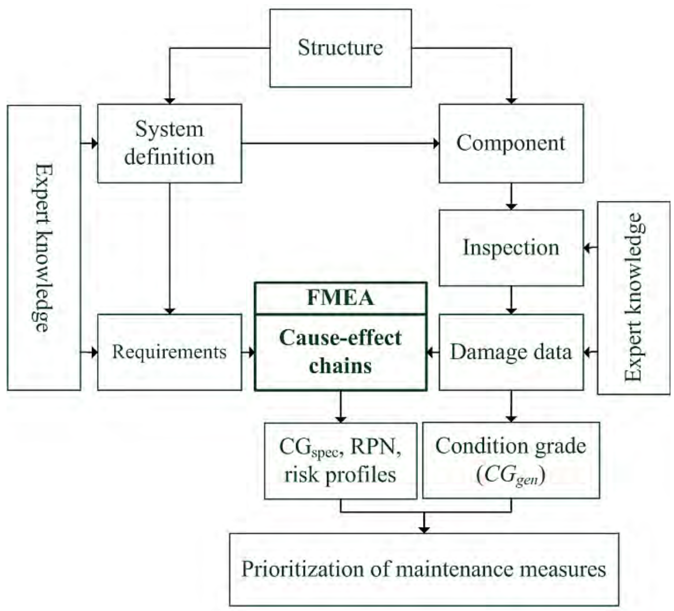

Figure 3: Implementation of FMEA in the condition assessment of navigational assets (Panenka & Nyobeu, 2018b).

Decision-making process can be considered as the choice, on some basis or criteria, of one alternative among a set of alternatives. Hence, a rational decision-making must be made on the basis of multiple criteria rather than a single criterion. This requires the assessment of various criteria and the evaluation of alternatives on the basis of each criterion and then the aggregation of these evaluations to achieve an overall relative ranking of the alternatives with respect to the pursued objective. Saaty (2008) has clearly recognized the inescapable necessity of making decisions based on several criteria and suggested that “*to make a decision, we need to know the problem, the need and purpose of the decision, the criteria of the decision, their sub-criteria, stakeholders and groups affected and the alternative actions to take*”. Again, Saaty (2008) contended that when trying to determine the best alternative or in the case of resource allocation, the appropriate share of limited resources must be supported by priorities for various alternatives. More specifically, if the main goal of our maintenance management strategy is to rank various failure modes for maintenance activities, the risk criteria/factors of the FMEA could be regarded as assessment criteria and the identified failure modes as various alternatives Thus, the decision of ranking several failure modes for repair and maintenance activities is decomposed into a multi-level hierarchic structure with the main objective at the top of the hierarchy, the decision criteria in the middle and the identified alternatives at the tail of the hierarchy (See Figure 4).

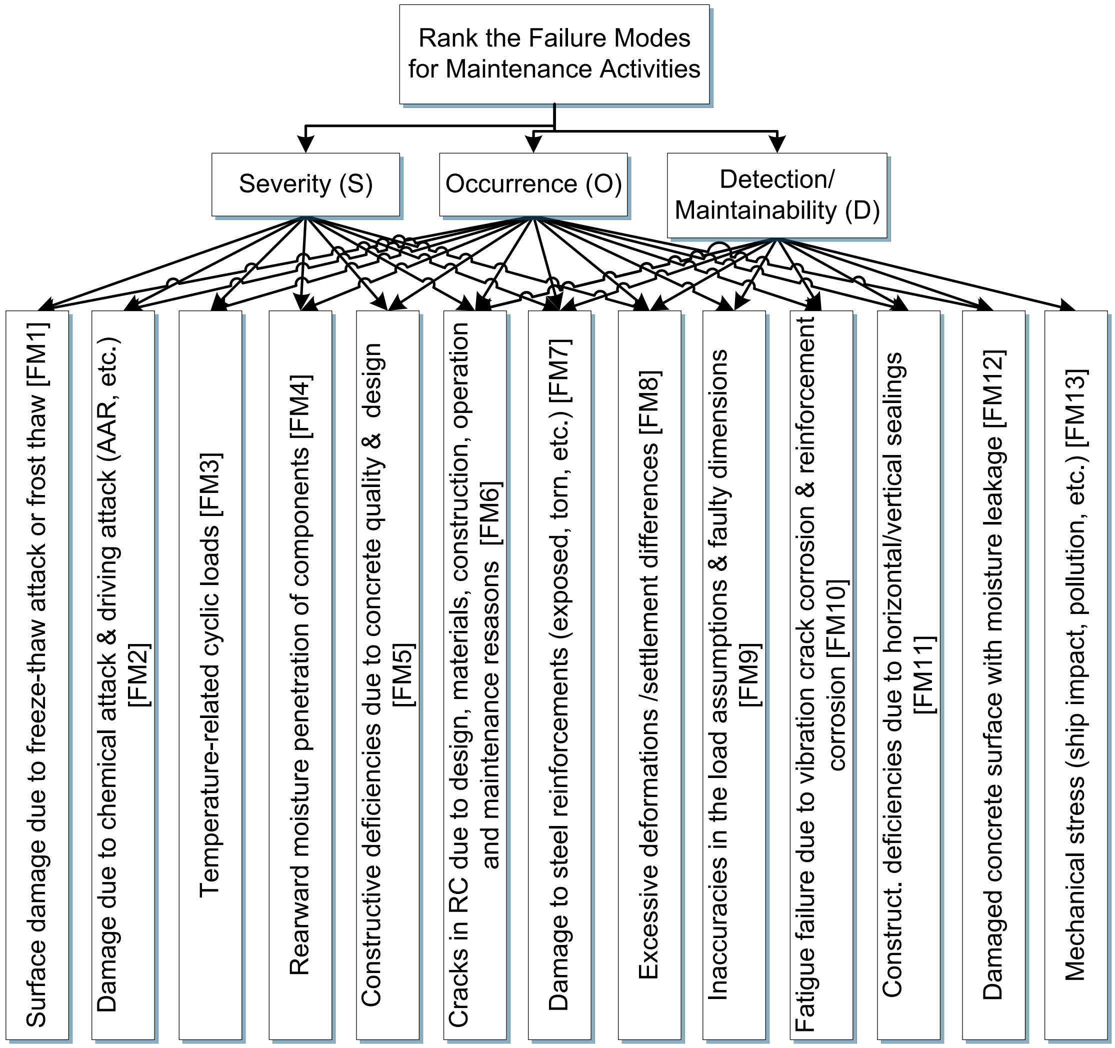

Figure 4: Decision hierarchy for ranking failure modes for repair and maintenance activities

In this context, a structured decision making procedure for the generation of overall priorities for alternatives can only be completed through the decomposition of the decision into the following steps.

a) Perform a FMEA in order to identify the potential failure modes;
b) Establishment of a hierarchy structure. The decision making process describing the problem to be solved, is decomposed into a hierarchy of goals, criteria sub-criteria and alternatives (i.e. failure modes). This is also called decision modelling and simply consists of building a hierarchy to analyze the decision (see Figure 4);
c) Construction of a set of pairwise comparison matrices for preference analysis. The relative importance of the criteria is pairwise compared with respect to the designed goal to derive relative priority weights for various criteria. The consistency of judgments is assessed to ensure that a reasonable level of consistency in terms of proportionality and transitivity is reached. Because the relative importance or weight of each criterion may differ from one to another, the relative priority of each criterion is derived through pairwise comparisons, using a numerical comparison scale, introduced by Saaty (1987) (See Table 2).

Intensity of Importance	Definition	Explanation
1	Equal importance	Two activities contribute equally to the objective
3	Moderate importance	Experience and judgment slightly favour one activity over another
5	Strong importance	Experience and judgment strongly favour one activity over another
7	Very strong or demonstrated importance	An activity is favoured very strongly over another; its dominance demonstrated in practice
9	Extreme importance	The evidence favouring one activity over another is of the highest possible order of affirmation
2,4,6,8	For compromise between the above values	Sometimes one needs to interpolate a compromise judgment numerically because there is no good word to describe it.
Reciprocals of above	If activity *i* has one of the above nonzero numbers assigned to it when compared with activity *j*, then *j* has the reciprocal value when compared with *i*	A comparison mandated by choosing the smaller element as the unit to estimate the larger one as a multiple of that unit
Rationals	Ratios arising from the scale	If consistency were to be forced by obtaining *n* numerical values to span the matrix
1.1 – 1.9	For tied activities	When elements are close and nearly indistinguishable; moderate is 1.3 and extreme is 1.9.

Table 1: Evaluation scale of relative preference for pairwise comparisons (Saaty, 2008)

d) Derive local priorities or relative preferences for the alternatives. Using the Saaty´s pairwise comparison scale, local priorities for alternatives are derived with respect to each criterion separately (following a similar process as in the previous step, i.e., compare the alternative pairwise with respect to each criterion). Once again the consistency of experts´ judgments is checked and adjusted if necessary.
e) Derive overall Priorities or weights for the criteria (Model Synthesis). Various alternative priorities, obtained during previous analysis are aggregated to generate a weighted sum that takes into account the weight of each criterion and establishes the overall priorities of the alternatives. Then, identify the alternative with the highest overall priority, which constitutes the most critical issue or the preferential choice for repair and maintenance activities.
f) Perform Sensitivity analysis. A study of how changes in the weights of the most influential criterion could affect the overall priority weights of various alternatives is done to understand the rationale behind the obtained results.
g) Making a final decision. Based on the synthesis results and sensitivity analysis, a decision can be made.

4. CASE STUDY

4.1 Assigning Priorities through Pairwise Comparisons

Waterway maintenance practitioners are faced with numerous questions, including what are the main failure modes that are likely to threaten the structural reliability of a navigational asset. They are also concerned with the influential relationship between these various failure modes. Thus, their prime objective is to establish a condition-risk-based maintenance strategy, which can be best used to address the huge burden of backlogged maintenance. Therefore, the case study discussed in this paper is an application of the AHP to solve the thorny problem of evaluating risk priorities of various failure modes and sharpening the focus on most pressing issues, in terms of their impact on the structural reliability of the asset, for maintenance activities. When selecting the most acute failure modes that urgently need to be maintained, the Severity (S) of the consequences of the failure mode, the probability of occurrence (O) as well as detection (D) of the failure mode are the main decision criteria to be considered. These criteria must be compared in order to derive their relative priorities (weights), and thus, determine their relative importance in meeting the overall goal of ranking various failure modes for maintenance.

Using the fundamental scale, comparisons that reflect the relative strength of preferences and feelings are made to construct the kernels of Fredholm operators from which ratio scales are derived in the form of principal eigenvectors or eigenfunctions (Saaty, 1987 and Saaty, 1990). These comparisons are fundamental in the use of AHP. We must first establish the relative priority of each of the three criteria by judging them in pairs for their relative importance, thus generating a pairwise comparison matrix. Judgments, which are represented by numbers from the fundamental scale, are employed to make the comparisons.

Examining the matrices below (see Table 2), one can note that a pair of elements (*ij*) in a level of the hierarchy are compared with respect to parent element in the level immediately above as a common property or criterion used to judge as to which one has it more and by how much. The typical way to phrase a question to fill an entry in the matrix of comparisons is: when considering two elements, *i* on the left side of the matrix and *j* on the top, which one contributes the most to the fulfillment of the overall goal, in other words, which one is considered more important in achieving the goal, described by the decision problem and up to what value (using the fundamental scale values from table 1)?. This gives us *aij* (or *aji*). The reciprocal value is then automatically entered for the transpose. The question asked in making a pairwise comparison can influence the judgments provided, and hence, also the priorities (see Table 2).

4.2 Assessment of various criteria/Deriving Priorities (Weights) for the Criteria

The importance of various risk criteria is pairwise compared with respect to the desired goal of deriving their relative priority weights (see Table 2). Since the proportion of inconsistency CR = 0.0036 is less than 0.1, we can assume that our judgments matrix is reasonably consistent.

	Severity (S)	Occurrence (O)	Detection (D) /Maintainability	**Normalised Priorities (Weights)**	**Idealised Priorities (Weights)**
Severity (**S**)	1	3	5	**0.6479**	**100%**
Occurrence (**O**)	1/3	1	2	**0.2299**	**35.48%**

Detection (**D**)	1/5	1/2	1	**0.1222**	**18.86%**
λ_{max} = 3.004	**CI = 0.0018**	**CR = 0.0036**			

Table 2: Pairwise comparison matrix for decision criteria with respect to the main objective

The normalized priorities may also be expressed in the ideal form by dividing each priority by the largest one, 0.6479 for Severity of the consequences of a failure. Basically, these results also indicate that the severity has 64.79% of the overall importance of the criteria, followed by the occurrence with 22.99% and detection (12.22%) respectively. Subsequently, the criteria Severity of consequences of various failure modes is regarded as the most important criteria in ranking various failure modes for repair and maintenance activities and the proportionate value of the occurrence and detection can then be estimated. The results of this criteria assessment clearly show that the probability of occurrence of the cause a failure is about 35.48% as good as the severity of its effects and the probability of a potential failure mode being detected is about 18.86% as good as the severity of its consequences.

4.3 Deriving Local Priorities (Preferences) for the Alternatives: Evaluation of alternatives on the basis of the assessed criteria

The next step consists of setting up comparison matrices for various failure modes (alternatives), which are compared with respect to each of the identified criterion. Since the derived priorities are valid only with respect to each specific criterion, they are often called local priorities. Thus, three more pairwise comparison matrices are generated to derive local priorities for each alternative based on the judged importance of one alternative over another with respect to a common criterion.

Severity	FM1	FM2	FM3	FM4	FM5	FM6	FM7	FM8	FM9	FM10	FM11	FM12	FM13	**Priority Vectors (Weights)**	**Rank**
FM1	1	2	3	3	1/3	1/7	1/7	1/5	2	1/3	3	2	3	**0.05785**	**7**
FM2	1/2	1	1/2	2	1/3	1/6	1/6	1/3	1/5	1/5	1/2	2	3	**0.02804**	**10**
FM3	1/3	2	1	3	1/2	1/5	1/5	1/3	1/3	1/3	2	2	4	**0.04144**	**8**
FM4	1/3	1/2	1/3	1	1/5	1/8	1/8	1/5	1/5	1/5	1/2	2	2	**0.02036**	**11**
FM5	3	3	2	5	1	1/3	1/3	3	1/3	3	3	5	7	**0.10159**	**4**
FM6	7	6	5	8	3	1	1/2	3	3	3	5	7	8	**0.18428**	**2**
FM7	7	6	5	8	3	2	1	3	3	2	5	8	8	**0.20406**	**1**
FM8	5	3	3	5	1/3	1/3	1/3	1	1/3	1/3	3	5	5	**0.07988**	**6**
FM9	1/2	5	3	5	3	1/3	1/3	3	1	3	5	5	7	**0.12346**	**3**
FM10	3	5	3	5	1/3	1/3	1/2	3	1/3	1	3	5	5	**0.09419**	**5**
FM11	1/3	2	1/2	2	1/3	1/5	1/5	1/3	1/5	1/3	1	2	2	**0.03139**	**9**
FM12	1/2	1/2	1/2	1/2	1/5	1/7	1/8	1/5	1/5	1/5	1/2	1	2	**0.01907**	**12**
FM13	1/3	1/3	1/4	1/2	1/7	1/8	1/8	1/5	1/7	1/5	1/2	1/2	1	**0.0144**	**13**

λ_{max}= 14.4037 CI = 0.11698 CR = 0.07499

Table 3: Pairwise comparison of alternatives with respect to the criterion Severity

The consistency test (CR = 0.07499 < 0.1) clearly shows that subjective judgments made during pairwise comparisons, are fairly consistency. Thus, the judgment matrix and the computed priority weights are quite reliable. Also, as it can be seen from the Table 4 (two right-most columns), the priorities for the identified failure modes with respect the severity of their effects, and thereafter, their respective contribution to the non-compliance with the design requirements of the system are computed. With priorities weights of 0.204, 0.184, 0.123, corrosion of steel reinforcements (**FM7**), reinforced concrete cracks (**FM6**) and inaccuracies in load assumptions and faulty dimensions (**FM9**) are respectively the three main failure modes, whose consequences can be detrimental to the structural reliability of the system. Likewise, comparison of various failure modes with respect to the probability of occurrence of the causes of potential failure modes as well as the probability of the failure being detected are displayed in the following tables 4 and 5.

Occurrence	FM1	FM2	FM3	FM4	FM5	FM6	FM7	FM8	FM9	FM10	FM11	FM12	FM13	**Priority Vectors (Weights)**	**Rank**
FM1	1	2	3	3	3	1/2	1/2	5	5	8	1/2	1/2	2	**0.09828**	**4**
FM2	1/2	1	3	3	1/3	1/3	3	3	4	6	1/2	1/2	2	**0.08912**	**6**
FM3	1/3	1/3	1	3	2	1/3	1/4	3	3	7	1/3	1/3	2	**0.05993**	**8**
FM4	1/3	1/3	1/3	1	1/3	1/3	1/3	2	2	2	1/2	1/8	1/3	**0.02945**	**10**
FM5	1/3	3	1/2	3	1	1/3	1/4	4	3	5	1/3	1/3	2	**0.06749**	**7**
FM6	2	3	3	3	3	1	2	6	8	8	3	1/3	3	**0.14795**	**2**
FM7	2	1/3	4	3	4	1/2	1	5	5	8	3	1/2	2	**0.12121**	**3**
FM8	1/5	1/3	1/3	1/2	1/4	1/6	1/5	1	2	2	1/4	1/8	1/3	**0.02131**	**11**
FM9	1/5	1/4	1/3	1/2	1/3	1/8	1/5	1/2	1	3	1/4	1/7	1/3	**0.01967**	**12**
FM10	1/8	1/6	1/7	1/2	1/5	1/8	1/8	1/2	1/3	1	1/7	1/8	1/5	**0.01235**	**13**
FM11	2	2	3	2	3	1/3	1/3	4	4	7	1	1/2	1/2	**0.09191**	**5**
FM12	2	2	3	8	3	3	2	8	7	8	2	1	3	**0.18193**	**1**
FM13	1/2	1/2	1/2	3	1/2	1/3	1/2	3	3	5	2	1/3	1	**0.05941**	**9**

λ_{max}= 14.4772 CI = 0.1231 CR = 0.07891

Table 4: Pairwise comparison matrix of alternatives with respect to the criterion Occurrence

Detection	FM1	FM2	FM3	FM4	FM5	FM6	FM7	FM8	FM9	FM10	FM11	FM12	FM13	**Priority Vectors (Weights)**	**Rank**

FM1	1	3	5	3	1/3	1/3	1/3	3	5	5	3	1/3	1/3	**0.07606**	**6**
FM2	1/3	1	3	3	1/3	1/4	1/3	1/3	3	3	1/3	1/4	1/5	**0.03677**	**9**
FM3	1/5	1/3	1	1/2	1/3	1/6	1/5	1/3	2	1/2	1/5	1/8	1/8	**0.01801**	**12**
FM4	1/3	1/3	2	1	1/3	1/6	1/4	1/3	1/2	3	1/3	1/6	1/8	**0.02307**	**10**
FM5	3	3	3	3	1	1/2	1/2	4	6	8	3	2	1/2	**0.11743**	**4**
FM6	3	4	6	6	2	1	3	4	8	8	3	1/4	1/2	**0.14141**	**3**
FM7	3	3	5	4	2	1/3	1	3	5	8	1/3	1/2	1/2	**0.10125**	**5**
FM8	1/3	3	3	3	1/4	1/4	1/3	1	3	4	2	1/3	1/3	**0.05414**	**8**
FM9	1/5	1/3	1/2	2	1/6	1/8	1/5	1/3	1	2	1/3	1/7	1/8	**0.01937**	**11**
FM10	1/5	1/3	2	1/3	1/8	1/8	1/8	1/4	1/2	1	1/4	1/8	1/8	**0.0151**	**13**
FM11	1/3	3	5	3	1/3	1/3	3	1/5	3	4	1	1/3	1/3	**0.06836**	**7**
FM12	3	4	8	6	1/2	4	2	3	7	8	3	1	1/2	**0.15362**	**2**
FM13	3	5	8	8	2	2	2	3	8	8	3	2	1	**0.17541**	**1**
						λ_{max}= 14.5436			**CI = 0.12863**			**CR = 0.08246**			

Table 5: Pairwise comparison matrix of alternatives with respect to the criterion Detection

The results of the pairwise comparisons with respect to the occurrence and detection are entered in a reciprocal comparison matrix as shown in Tables 4 and 5. With values of the consistency ratio estimated at 0.07891 and 0.08246 respectively for the criteria occurrence and detection, it can be undoubtedly inferred that these judgment matrices are consistent enough. It is however important to highlight that the failure modes **FM12, FM6, FM7** and **FM1** are the main deterioration processes that affect the structural reliability of reinforced concrete navigational assets. On the other hand, failure modes **FM13, FM12**, **FM6**, **FM5** and **FM7** can be easily detected, in comparison to failure modes **FM10**, **FM3** and **FM9** that are difficult to detect, and whose consequences are likely to significantly alter the structural reliability of the asset.

4.4 Model Synthesis: Aggregation of the local priorities to derive overall priorities of alternatives with respect to the problem to be solved

To determine the final priorities of various failure modes with respect to the main goal of the present study, which consists of ranking failure modes for repair and maintenance activities, individual judgments, made at the risk criteria and alternatives levels must be aggregated. Thus, a synthesis of various analyses is carried out by multiplying each ranking by the priority of its criterion or sub-criterion and adding the resulting weights for each alternative to obtain the final priorities of the 13 failure modes. In other words, to aggregate the local priorities of the 13 failure modes, an aggregated matrix is developed by calculating the arithmetic mean according to Forman and Peniwati (1998) (See Table 6 below).

	Severity	Occurrence	Detection	Global Priority Weights	Rank
Criteria Priority Weights (CW)	**0.6479**	**0.2299**	**0.1222**		
Surface damage due to repeated cycles of freezing and thawing **(FM1)**	0.05785	0.09828	0.07606	**0.069**	**6**
Damage due to chemical attack & driving attack (e.g., AAR, etc.) **(FM2)**	0.02804	0.08912	0.03677	**0.043**	**11**
Temperature-related cyclic loads **(FM3)**	0.04144	0.05993	0.01801	**0.043**	**12**
Rearward moisture penetration of components **(FM4)**	0.02036	0.02945	0.02307	**0.023**	**13**
Constructive deficiencies associated with concrete quality & concrete design methodology **(FM5)**	0.010159	0.06749	0.11743	**0.096**	**3**
Cracks in RC due to design, materials, construction, operation and maintenance reasons **(FM6)**	0.18428	0.14795	0.14141	**0.171**	**2**
Corrosion of steel reinforcements (exposed, torn, etc.) **(FM7)**	0.20406	0.12121	0.10125	**0.172**	**1**
Excessive deformations /settlement differences **(FM8)**	0.07988	0.02131	0.05414	**0.063**	**8**
Inaccuracies in the load assumptions & faulty dimensions **(FM9)**	0.12346	0.01967	0.01937	**0.087**	**4**
Fatigue due to vibration cracks & corrosion of embedded steel reinforcements **(FM10)**	0.09419	0.01235	0.0151	**0.066**	**7**
Constructive deficiencies associated with horizontal and vertical sealings **(FM11)**	0.03139	0.09191	0.06836	**0.05**	**9**
Damaged concrete surface with moisture leakage **(FM12)**	0.01907	0.18193	0.15362	**0.073**	**5**
Mechanical stress (ship impact, pollution, etc.) **(FM13)**	0.0144	0.05941	0.17541	**0.044**	**10**
	Overall CR of the hierarchy = 0.05849				

Table 6: Synthesis of the model priorities

With an overall inconsistency ratio (CR) of about 0.05849 < 0.1, the correctness and the consistency of the given pairwise comparisons is quite satisfactory. Based on the calculation of these overall/combined priority weights, different alternatives (failure modes) can be sorted according to their weight values, as shown in table 6. The analysis of the overall priority weights puts in evidence that reinforced concrete cracks and corrosion of steel reinforcements are the main deterioration processes that have significant effects on the structural requirements of navigational assets. It can also be noted that construction deficiencies, associated with concrete quality and design methodology must be given a great consideration in the design and construction phase, in order to avoid their further consequences that are likely to substantially alter the durability of assets.

4.5 Sensitivity Analysis

In many areas where the optimization of processes or systems is granted a great consideration, the question of sensitivity of results has always been at the centre of concern. Sensitivity analysis enables decision makers to improve the credibility of their analytical model by providing appropriate answers to "*what if*" questions and by quantifying the robustness of the optimal solution under variations in the problem parameters (Erkut and Tarimcilar, 1991). The specific objective of the sensitivity analysis is to check whether few changes in the judgment evaluations may result to significant modifications in the overall priority ranking. Thus, sensitivity analysis is used to investigate the robustness of the overall priority weights of alternatives to changes in the priorities of the criteria at the level immediately below the main goal of the analysis.

More specifically, although the criteria assessment undoubtedly suggests that the "severity" is the most important criteria (see Table 2) in ranking various failure modes, the overall ranking of alternatives is likely to change in accordance with shifts in analyst logic. Basically, each criterion is characterized by an important degree of sensitivity, i.e. the ranking of all alternatives may change dramatically over the entire weight range (Erkut and Tarimcilar, 1991). Therefore, by changing the priority weights (relative importance) of various criteria, a series of sensitivity analysis can be performed to explore the robustness of the current solution to potential shifts. However, it is important to stress that the sensitivity analysis, proposed in this paper is only relevant to the priorities of the three criteria, selected in this study. Also, in answering the following questions, different scenarios are simulated:

- what if the priority weights of various criteria are given the same value?
- what would be the best alternative if the relative importance of a single criterion is changed?

In assessing the impact of the change of a single criterion on the overall ranking of alternatives, only the "main effects" must be considered, as suggested by Bevilacqua and Braglia (2000). In other words, "interaction effects" of the changes made to the other two weights are ignored. This simplification is introduced by Bevilacqua and Braglia (2000), who contended that the final solution was mainly sensitive to changes in the priorities at the highest level of the hierarchy. Moreover, it is assumed that the AHP is used as a decision-making tool and that the decision maker is merely interested in ranking various alternatives. Hence, we are especially interested in the sensitivity of the alternative with the highest ranking.

4.5.1 Assigning the same value to the priority weights of the three criteria

Table 7 shows the results of the sensitivity analysis, based on the main assumption that the three criteria have the same relative importance and are assigned the same priority weight of 1/3. This consideration also leads to a reduction of about 50% of the relative importance of the criterion severity. It can also be observed that by adopting large changes of the weights of the first criteria it is possible to substantially alter the overall priority weights and the final ranking of several alternatives (**FM8, FM10, FM12 and FM13**). In addition, the initial ranking between the alternatives is not preserved, although reinforced concrete cracks and corrosion of steel reinforcements remain the relevant failure modes. Also, a shift of about 17% is observed between the initial priority weight of the failure mode **FM7** (0.172) and the current value of 0.1422.

	Severity (S)	Occurrence (O)	Detection (D) /Maintainability	**Priority Weights**	**Rank**
CW	**0.3333**	**0.3333**	**0.3333**		
FM1	0.0579	0.0983	0.0761	**0.0774**	**6**
FM2	0.0280	0.0891	0.0368	**0.0513**	**10**
FM3	0.0414	0.0600	0.0180	**0.0398**	**12**
FM4	0.0204	0.0295	0.0231	**0.0243**	**13**
FM5	0.0102	0.0675	0.1174	**0.0955**	**4**
FM6	0.1843	0.1480	0.1414	**0.1579**	**1**
FM7	0.2041	0.1212	0.1013	**0.1422**	**2**
FM8	0.0799	0.0213	0.0541	**0.0518**	**9**
FM9	0.1235	0.0197	0.0194	**0.0542**	**8**
FM10	0.0942	0.0124	0.0151	**0.0405**	**11**
FM11	0.0314	0.0920	0.0684	**0.0639**	**7**
FM12	0.0191	0.1820	0.1536	**0.1182**	**3**
FM13	0.0144	0.0594	0.1754	**0.0831**	**5**

Table 7: Scenario 1: same value for the priority weights of the three criteria

4.5.2 Adopting different values for the priority weights of the three criteria

Similarly, the table 8 below illustrates the outcome of the sensitivity test, taking into account a reduction of about 25% of the initial value of the priority weight of the severity to 0.4859. It is apparent from these analyses that the values of the overall priority weights of various failure modes are affected by a change of about 5 to 8% of their initial values. More importantly, it can be seen that the rank of various failure modes (**FM2, FM3, FM4, FM5 and FM11**) has not been affected by the modification of the initial weight of the criteria severity. On the other hand, cracks in reinforced concrete, damages to steel reinforcements and Construction deficiencies in relation to concrete quality and concrete design methodology remain the essential failure modes that must be prioritized for maintenance activities.

	Severity (S)	Occurrence (O)	Detection (D) /Maintainability	**Priority Weights**	**Rank**
CW	**0.4859**	**0.257**	**0.257**		

FM1	0.0579	0.0983	0.0761	**0.0729**	**5**
FM2	0.0280	0.0891	0.0368	**0.046**	**11**
FM3	0.0414	0.0600	0.0180	**0.0402**	**12**
FM4	0.0204	0.0295	0.0231	**0.0234**	**13**
FM5	0.0102	0.0675	0.1174	**0.0969**	**3**
FM6	0.1843	0.1480	0.1414	**0.1639**	**1**
FM7	0.2041	0.1212	0.1013	**0.1563**	**2**
FM8	0.0799	0.0213	0.0541	**0.0582**	**8**
FM9	0.1235	0.0197	0.0194	**0.070**	**6**
FM10	0.0942	0.0124	0.0151	**0.0528**	**10**
FM11	0.0314	0.0920	0.0684	**0.0564**	**9**
FM12	0.0191	0.1820	0.1536	**0.0955**	**4**
FM13	0.0144	0.0594	0.1754	**0.0674**	**7**

Table 8: Different values for the priority weights of the three criteria

As one can see, only by adopting large changes of the weights of the criterion severity, it becomes possible to significantly alter the values of the priority weights of various failure modes, although a reduction of the main criterion severity of about 25% results in a shift of the first position in final ranking of different alternatives. These findings clearly show an intrinsic robustness of the final priority weights, developed by means of the AHP method, considering sensitivity of the final ranking of alternatives to changes in the weights of the criteria in the second level of the decision hierarchy.

5. DISCUSSION AND CONCLUSION

This paper presents a methodology for ranking various failure modes and selecting most crucial failures that could alter the structural reliability of a navigational asset for repair and maintenance activities. In a context where German waterway infrastructure are ageing and faced with various deterioration mechanisms, the emerging risk of infrastructure failure is increasingly becoming an issue of great concern. Therefore, a rational decision making about repair and maintenance activities, supported by key figures describing the structural effects of various degradation mechanisms on the structural reliability of assets, is a foremost imperative. In addition, maintenance practitioners are often confronted during cyclic visual inspections with challenging decision making about which failure modes are likely to undermine the structural reliability of the asset. Yet, risk assessment techniques play a vital role in maintenance decision making given that these techniques can be used to systematically identify, analyze, evaluate current deterioration mechanisms and mitigate their potential risks with respect to the consequences of an infrastructure failure. Using the Analytical Hierarchy Process, primarily developed by Saaty (1990), priority weights for ranking various failure modes are derived based on various criteria of the Failure Mode and Effects Analysis (FMEA) and taking into account uncertainties, associated with the subjectivity of expert knowledge. In comparison to other traditional methods of solving the MCDM problem, the proposed methodology does not only assess multi-criteria decision problems in a more objective way by avoiding subjective effects on the priority weights, but it simultaneously serves for qualitative and quantitative analysis of available data. The ranking of priority weights of various failure modes, obtained from the FMEA-AHP-model is depicted in Table 8 as follows: **FM6 > FM7 > FM5 > FM12 > FM1 > FM9 > FM13 > FM8 > FM11 > FM10 > FM2 > FM3 > FM4**. Also, the findings of this study have highlighted the existence of four main categories of failure modes (See Figure 5). The first category that include the failure modes **FM1, FM2, FM3** and **FM4** with a relatively moderate probability of occurrence, which are easily detected and whose consequences may not have further incidence on the structural reliability of the asset. While immediate consequences of these categories of failures on the structural reliability of an asset are very limited, their long term effects are likely to hinder the durability of an asset.

The second category consists of failure modes with a very high probability of occurrence, which are likely to be very easily detected and whose may have severe and unbearable consequences for the structural reliability of the facility. Including the failure modes **FM5**, **FM6** and **FM7**, this second group represents the main challenges that are facing maintenance practitioners and that urgently need to be addressed because of their effects on the load-carrying capacity of the structure. The third category of failure modes merely consists of failures **FM8**, **FM9**, and **FM10** with a very low probability of occurrence, which are often very difficult to detect and whose consequences can be detrimental to the structural reliability of the asset. The fourth category is composed of failure modes **FM11**, **FM12** and **FM13** with a very high probability of occurrence, which can be very easily detected and whose consequences are relatively low. These failures are more likely to have long term effects on the durability of the infrastructure. From the perspective of combined effects of various failure modes, it is however important to stress that long term effects failure modes of the first and fourth categories may inevitably lead to the loss of the structural reliability of the infrastructure. For instance, an asset may be weakened by constructive deficiencies associated with horizontal and vertical sealings that could subsequently result in excessive deformations/settlement differences.

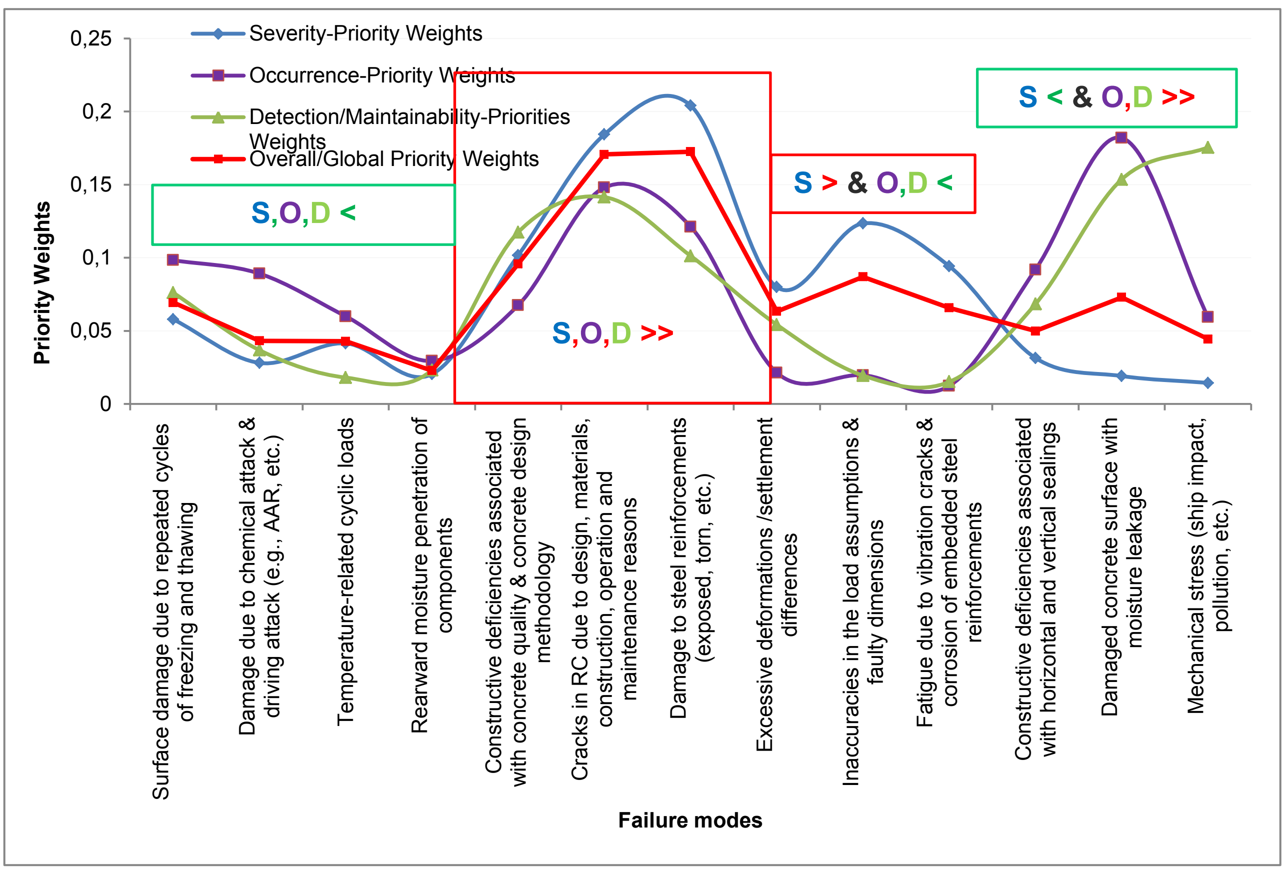

Figure 5: Computation of priority weights of various failure modes

Some limitations of the proposed approach need to be emphasized. The approach largely depends on expert knowledge and damage data that entail a considerable amount of uncertainties. Moreover, the AHP methodology is considered costly, in terms of time given the number of cluster matrices required in the exercise. Nevertheless, the proposed approach attempts to address an important gap in practice by proposing a structured framework for ranking and selecting various failure modes for repair and maintenance. In addition, the selection models accounts for organizational capabilities, defined through the decision elements. The alternative risk assessment model, based on Analytical Hierarchy Process (AHP) serves as a tool to prioritize the Risk Priority Number (RPN) for each failure mode assessed and allow the identification of the most critical ones in the system. This aspect can ultimately be used as basis for strengthening the investment decision in favor of improving the reliability of the entire system. In general, our results indicate that the severity of potential consequences of a failure mode is the essential criterion in identifying the failure mode that posed the highest risk, and thus, that urgently need to be maintained.

To fully capture the linguistic vagueness behind the development of our comparison judgment matrices for the derivation of crisp priority weights, a fuzzy AHP may be used in future research studies. In addition, the Technique for Order Preference by Similarity to Ideal Solution (TOPSIS) may be employed for evaluating and ranking various failure modes. Also, future works may focus on developing both qualitative and quantitative approaches, supported by Multi-Criteria Decision Making (MCDM) methods for the investigation of the emerging risk of infrastructure failure (Schmidt-Bäumler, 2017). A first step could consist of using the fuzzy logic approach, proposed by Bowles & Peláez (1995) in their study, to correct or mitigate the effects of a failure to be prioritized even though the available information might be vague, ambiguous, qualitative or imprecise. The Findings of such research projects could be used to strengthen the expressiveness as well as conclusiveness of the current condition grades and enhance decision making about prioritization of maintenance activities.

ACKNOWLEDGMENTS

The authors would like to thank the BMVI Network of Experts, initiated by the German Federal Ministry of Transport and Digital Infrastructure (BMVI) for financing and supporting this work.

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ADVANCES IN THE METHODOLOGY FOR THE INLAND WATERWAYS CLASSIFICATION FOR SOUTH AMERICA

by

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Abstract

At the present time, there is no harmonized approach or a set of data and information on the navigation conditions of the South American waterways which would allow to assess realistically the current and potential capacity of the network for mobility of goods or passengers. In addition to the practical limitations that this entails for an everyday use of inland water transport, this situation also negatively affects the planning and policy making, when it comes to the national and regional policies aiming to increase the use of inland navigation in the region.

Since October 2016, ECLAC and PIANC are working together in close collaboration with the experts from South American countries on a common classification for the inland waterways. The first results of this work, including the definition of the objectives, scope, general structure and parameters for the classification, are presented in this paper.

Introduction

The inland waterways' network in South America is known for having one of the highest density and widest geographical coverage in the world. It is composed of several independent inland waterway systems, each in different level of development, including the Amazon basin, the Paraguay-Parana, the Paraná-Tietê, the Orinoco, the Araguaia-Tocantins, the São Francisco, the Plata Basin, the Essequibo, the Magdalena, the Uruguay and that of Xingu. The Amazon basin extends through the Solimões River and connects with 5 countries (Brazil, Colombia, Bolivia, Ecuador and Peru) and their tributaries. In this way, this basin offers a connection to the Atlantic for the interior parts of South America. Complementary to the Amazon basin is the HPP, whose backbone, the Paraná River, is limited upstream in its navigation by the Itaipú dam. Its zone of influence extends through Argentina, Bolivia, Brazil, Paraguay and Uruguay. The Paraguay River, which rises in the Mato Grosso plateau and ends in Confluencia, it contains the main port terminals of Paraguay, the two of Bolivia and the area of influence of Brazil's Mato Grosso. The Uruguay River, which is limited in its navigation to the north by the Salto Grande dam, allows the navigation to the south of it, through the Río de la Plata towards the Atlantic Ocean (Jaimurzina and Wilmsmeier, 2017).

Despite this endowment in river density, the South American fluvial network is not being sufficiently exploited for the mobility of cargo and people. As an example, in Colombia less than 1% of the cargo is transported by waterways and out of a total of 24,274 km of navigable waterways, only 18,225 km are actually used. In Brazil, in turn, of a fluvial network of approximately 63,000 km and 42,000 potentially navigable kilometers, only 20,000 are currently used for navigation. (Jaimurzina and Wilmsmeier, 2017).

It is estimated that the fluvial transport of greater volume takes place in the Amazon basin (21 million tons per year), followed by the Paraguay-Paraná Waterway (Hidrovía Paraguay-Paraná - HPP), with 14 million tons per year, the Araguaia-Tocantins (4 million tons), and Magdalena and Paraná-Tietê (2 million tons each) (Jaimurzina and Wilmsmeier, 2017). In addition to the volumes of national and foreign trade flows that are transported through some of the region's waterways, in a large part of the rivers and basis of the region, the local commercial and passenger flows of river transport, are of greater importance than those corresponding to international or transboundary flows, and are also greater than the general perception of fluvial activity (Bara et al., 2006).

It is generally considered that the topography and the uneven density of hydrographic systems in South America prevent the greater use of the inland navigation. The unpredictability of weather conditions, significant changes in water levels and obstacles to navigation (such as sandbanks and palisades), for instance, prevent or stop transport temporarily. Furthermore, problems of draft limitations and predictability exist in large parts of the fluvial network.

These infrastructural challenges are not likely to be addressed by the business-as-usual transport and infrastructure policies. The persistent gap between the needs for infrastructure investments and the amount of public and private investment observed over the last two decades (Sánchez y al, 2017) is amplified by the road-dominated modal split in the transport infrastructure investments (Jaimurzina and Wilmsmeier, 2017), leaving the development of inland waterways in a margin of many transport and logistics policy interventions.

What is needed is a profound shift towards a new generation of integrated and sustainable transport and logistics policies, based on a comodal approach to transport operations and a balanced and realistic analysis of the benefits and limitations of each mode of transport leading to an overall greater efficiency, sustainability and resilience of the entire transport system. These new policies require new tools and instruments, going beyond the traditional perspectives and criteria and filling the gaps in the elements available for an integrated and sustainable approach. One of these instruments, as far as inland navigation is concerned, is a common classification of inland waterways, as a means of identifying their capacity for the transport of goods and passengers.

At the present time, there is no harmonized approach or a set of data and information on the navigation conditions of the South American waterways which would allow to assess realistically the current and potential capacity of the network for mobility of goods or passengers. In addition to the practical limitations that this entails for an everyday use of inland water transport, this situation also negatively affects the planning and policy making, when it comes to the national and regional policies aiming to increase the use of inland navigation in the region. In this sense, the Plan Maestro Fluvial in Colombia highlighted that one of the main difficulties for estimating the needs of the investment is the absence of an updated and detailed inventory of the state of the river infrastructure, which can serve as a basis for an estimate of the costs of infrastructure works (Plan Maestro fluvial, Colombia, 2015).

This leads to a situation where most of the analytical and policy work on infrastructure gap in South America, either leaves out waterborne transport, focusing on the road and rail sector, or is based on very generalized assumptions, which undermines its utility in practice (ITF Transport outlook). The mere comparison of the levels of investment in road, rail and waterborne transport gives limited information, if there are no reliable estimates for the infrastructure investment needs for all modes of transport. By the same token, the potential benefits of infrastructure works on a part of an inland waterway should not be considered in isolation from the state of the rest of the network and any feasibility analysis would be enhanced if complemented by the assessment of the impact of works on the overall capacity of the network.

In other regions of the world, a common classification of inland waterways was instrumental in identifying the core and secondary IW network and its missing links as well as to monitor its development and to evaluate to what extent the implemented infrastructure projects enhanced the capacity of the network (Jaimurzina and Wilmsmeier, 2016). A similar tool could be developed for South America, incorporating additional policy concerns, such as preoccupation with the level of logistics and mobility services and a greater sustainability in providing infrastructure services.

It is in this context, that ECLAC and PIANC, in consultation with the South American and international experts, have launched a joint initiative on elaboration of a common classification for South America which would promote a more efficient, transparent and sustainable use of inland water transport and logistics services, in general. A common ECLAC/PIANCL informal working group was set up in 2016, following the recommendations of the

ECLAC/PIANC/ANTAQ Seminar on Inland navigation and a more sustainable use of natural resources: networks, challenges, and opportunities for South America (Rio de Janeiro, 19 October 2016).

The goal of the working group was to a) provide a forum for initial technical meetings between South American experts, including also international experts, on the future inland waterways classification for South America; b) collect and analyze information and data on inland waterways characteristics, fleet (for inland, recreational and seagoing vessels), intensity of traffic and other relevant factors for the elaboration of technical and operation parameters, harmonized at the regional level; and c) formulate an advanced draft of the technical and operation parameters for the classification and present the preliminary results of such classification for the (selected) countries of the region. In doing so, the Working Group was to collect recent development and case studies from different regions and countries on classification standards and to review the standards, and best practices in this field in order to recommend them if and when appropriate as part of the final report.

The following paragraphs summarize the results of the Working group's technical meetings and teleconferences on the main issues related to the classification, including:

1) Objectives and scope of the classification;
2) Overview of the existing classification systems in South America;
3) First methodology proposal for the inland waterway classification for South America
4) Pending issues and future work.

I. OBJECTIVES AND SCOPE OF THE POSSIBLE SOUTH AMERICAN CLASSIFICATION SYSTEM FOR INLAND WATERWAYS

The analysis of the background information of inland waterways classification in South America identified the several elements of relevance for the classification work.

It was observed that that South America, as of yet, has not been able to take full advantage of its extensive system of naturally navigable waterways and in making them an integrated part of the region's transport network to cater for the ever-increasing demand for cargo and human mobility. In this context, a common inland waterways classification could provide a tool for assessing the current status of the existing waterways and their current and potential capacity to integrate into the national and regional logistics chains, helping inter alia the region to transition to a more sustainable use of its national resource by implementing a more sustainable logistics system.

It was noted that the introduction of the River Information Services and other technological developments in South American inland navigation provided useful tools and information for elaborating and implementing a common regional classification.

Finally, it was also observed that the increasing preoccupation with the sustainability issues, especially environmental sustainability, could and should also be channeled through the classification work.

The exchanges on the benefits of the classification identified various possible positive impacts of the classification both for public and private sector.

Table 1
Benefits of the common classification system from the public policy and user perspectives

From the public policy perspective	From the private sector/user perspective
❑ Measuring and monitoring the state of IW infrastructure.	❑ Information on the navigation conditions ❑ Ease of navigation ❑ Security of navigation

❑ IW inventory which facilitate inter-modal integration ❑ Basis for estimating the investment gap, maintenance needs and impact of new investments, ❑ Facilitate access to financing. ❑ Incorporate sustainability concerns, ❑ Common basis for bilateral and regional agreements.	❑ Standards for RIS ❑ Better conditions for industrial development (naval constution) ❑ Parameters for estimating costs and benefits of investments in fleet, new infrastructure and maintenance

Source: ECLAC/PIANC Working Group on PIANC/InCom/ECLAC on Policies and Development of the Inland Waterway Classification for South America, 2017*.*

There has been an emphasis on the link between the classification and the financing, both public and private, through identification of the viable infrastructure projects, a more comprehensive view of the impact of the individual projects on the overall capacity of the waterway network and identification of the specific interventions which would result in a qualitative leap in the inland navigation operations.

The intervention of the public sectors' representatives and from ECLAC also highlighted that the classification could offer a concrete way of incorporating sustainability concerns in the management and development of the inland waterways, preserving and capitalizing upon the environmental and other benefits of the inland navigation.

Drawing on these elements, the expert and policy discussions organized by ECLAC and PIANC in 2016 and 2017, together with a survey of the waterborne transport experts in the region, allowed to identify the following four elements at the main objectives of a common regional classification of South American inland waterways:

- Supporting inland waterways and, more generally, transport policies and projects in Infrastructure development and operation, including planning, monitoring and identifying missing links and bottlenecks that should be prioritized;
- Increasing safety and ease of navigation by ensuring the orderly and efficient control and maintenance of waterways;
- Facilitating the planning of regional integration projects;
- Achieving a more sustainable use of inland waterways (and transport in general).

It was recommended that the classification should facilitate the decision- and policy making processes both by public and private sector to promote a greater use of inland navigation in the region. To the extent possible and in a pragmatic way, it should offer both the elements needed to estimate the extent of the public infrastructure investments (to upgrade or maintain the level of services), and a minimum information on the operational requirements for the benefit of the private sector and other users of inland navigation.

Finally, as far as broader requirements for the classification are concerned, it was recommended that, similar to the existing international classifications, the South American classification satisfy the following criteria:

- o Introduce a hierarchy of classes, hereby guaranteeing that a vessel or convoy normally operating on waterways of one class could be used on waterways belonging to a higher category without restriction as to the parameters covered by the classification;
- o Be forward-looking in its design, specifying the parameters to be complied with when constructing new or modernizing existing inland waterways with the objective of contributing to the sustainable development of the entire region, that is, to establishing an integrated, comodal and sustainable transport network at the national and regional level;
- o Be based on the specific conditions of navigable waterways in South America and, in this sense, be able to:
    - Be applied to the widest possible area of South America;

- Adapt to future developments in the technology of inland navigation.
- Incorporate waterways of diverse characteristics, given the important social and economic function of some sections at the local level.
- Sufficiently dynamic and flexible to accommodate the diversity of navigation conditions related to hydrography and climate.

o Be applied easily and in a simple way and be easily understood by different stakeholders

II. EXISTING INLAND WATERWAYS CLASSIFICATIONS IN SOUTH AMERICA

Bearing in mind the requirements and objectives of the future South American classification of inland waterways, the Working Group proceeded to analyzing the existing international and regional best practices in the classification area.

Prior to the creation of the Working group, the ECLAC/PIANC Joint Working Paper on Classification of Inland Waterways concluded that, while the existing international classification, i.e. the UNECE/CEMT Classification for European inland waterways, demonstrated practical impact and various uses of inland waterways classification for infrastructure development and for defining the basic regulatory framework for inland navigation, its technical and operational criteria for classification did not correspond to the characteristics of the inland waterways in South America.

Following this analysis, one of the first tasks the Working Group was to gather information about the existing inland waterways classifications at the national level, existing in South America. As stated previously, the South American countries in this region have not developed a clear and practical IW classification, except for Brazil and Colombia. The main characteristics and lessons learned of these national experiences are summarized below.

a. Inland waterways classification in Brazil[1]

The Brazilian Waterway Classification System was developed by the Brazilian Departamento Nacional de Infraestrutura de Transportes (DNIT), and was codified on 13 September 2016 in Administrative Bulletin No 172, Portaria No 1.635 (see Appendix A for the Brazilian Policy).

This Codified Policy was based on "studies and plans already published by Agencies and Organizations in the Waterway Sector and more than 40 years of surveys about the principle rivers that are currently in the Federal System (SFV)". The Brazilian Waterway classification system defines the design vessel dimensions for length and width, and includes a parameter in the system for minimum operational waterway depth (not vessel draft).

The classification system is presented in Table 2 and 3.

Table 2: Classes in the Brazilian System of Design Vessels

Class	Maximum Width (B), m	Length (L), m
I	48	280
II	33	210
III	25	210
IV	23	210
V	16	210
VI	16	120
VII	12	140
VIII	12	80
IX	12	50

[1] The summary of the Brazilian system is based on a technical report, prepared by Calvin Creech, U.S. Army Corps of Engineers, in close consultations with the national experts in Brazil.

Table 3: Sub-Classes (Categories) in the Brazilian System based on Waterway Depth

Category	Minimum Operational Depth (P), m
Special	P > 3.50
A	3.50
B	3.00
C	2.50
D	2.00
E	1.50
F	1.00

Source: Administrative Bulletin No 172, Portaria No 1.635

In the Brazilian system, the width of the waterway itself (for straight reaches) is based on the following formulas:

- One-lane traffic the Waterway Width (W) = 2.2 x Maximum Vessel Width (B)
- Two-lane traffic the Waterway Width (W) = 4.4 x Maximum Vessel Width (B)

Additional waterway widths for curve sections are not codified in the Brazilian Administrative Bulletin Nº 172, Portaria Nº 1.635, but in practice, the following formula is generally used for additional widths in curves:

$$B_m = B + \frac{L^2}{2R}$$

Where, B_m = Channel Width in a meander or curve
B = Channel Width in a straight reach
L = Length of the design vessel or convoy

Additional design elements of the Brazilian system include:

- A curve in the waterway is defined when the radius is less than 10 x the tow length (L)
- A curve cannot have a radius less than 4 x the tow length
- Distances between curves must be a minimum of 5 x the tow length
- Dredging sites require minimum side slopes of 1:8 for alluvial channels
- Rock excavation sites require minimum side slopes of 1:1

The classification has been applied by DNIT on various waterways since the adoption of the classification. For example, in Brazilian Administrative Bulletin No. 021 Portaria No 158 (30 January 2017) the Madeira River was established as a Class II-A Waterway between Porto Velho and the confluence with the Amazon River. In the same Bulletin the Rio Paraná was established as a Class V-A Waterway between Foz do Iguaçu, PR and São Simaõ, GO; and is established as a class VII-A between the confluence with the Rio Tietê and the Três Irmãos lock. In both river systems, it is noted that these specifications do not apply for the waterways during high flows (presumably to allow for larger convoy configurations).

At the present moment, the Brazilian system represents the most advanced inland waterways classification in South America.

b. Inland waterways classification in Colombia[2]

There have been two attempts to classify the waterways in Colombia: by the DNP (Departamento Nacional de Planeación) and Ministry of Transport in 1994 and by the Ministry of Transport in 2000.

The classification of 1994 categorizes the national fluvial network in “primary” or “secondary”, depending if the waterway has an important flow of cargo or is mainly dedicated to regional activities (to take also into account the social benefit of a waterway). It should be highlighted that already from these first classification efforts, the need of a social component was found necessary when classifying a waterway in a Latin American country, and not only make a classification based on economic variables or volumes of transported cargo.

The Manual of Navigable Rivers published by the Ministry of Transport in 2000 is the second attempt of classification. Despite the enormous efforts of this document to formalize and summarize the restrictions in navigation in the waterways of the country, the information presented in schematic maps and tables is not precise. The waterways are categorized in this document in “major” (which can be “permanent” of “transitory” depending if the river is navigable the whole year or is interrupted in the summer periods) and “minor” navigation. And this classification is the result of extrapolating for waterways the classification in “major” and “minor” vessels made by the ministry of Transport in 1999 for the Colombian fleet. “Major vessels” are those with a DWT higher or equal to 25 Ton; and therefore, “major waterways” are those with capacity to allow the traffic of “major vessels”.

These classifications have been used consistently up to nowadays, including the most recent study on this topic, the Fluvial Master Plan of 2015 (“Plan Maestro Fluvial de Colombia 2015”, PMF). However, these classifications do not follow a rigorous method with objective parameters used in the analysis, ending up in a classification very much dependent on qualitative interpretations, rather than a formal classification using quantifiable criteria.

The concerns and limitations of the existent classifications are already recognized by the fluvial sector, which would embrace a new more formal classification knowing the benefits this brings with a stronger and clear frame to perform their profession.

The information available at present for a new classification of the waterways in the country is incomplete, outdated and spread in several sources. A first step forward would be the collection of data about the existent fleet and the navigation conditions at present. Although it is not strictly necessary to know all this information to propose a classification system, it is important and it would be useful to know what type of vessels (and quantity) are used in Colombia at present and know about the status of the waterways at present. Besides, this information will be necessary anyway later on to classify the waterways in all the basins of the country. The PMF 2015 also stresses that a detailed and complete update of the information about the Colombian fluvial fleet and the status of the waterways is highly recommended. The way how this information is collected and sorted should be done in such a way that it can also be used later on for the implementation of a RIS in Colombia, inexistent today.

c. Other inland waterways classifications in South America.

While there were no other examples of national classification in South America, identified by the group, two additional elements were highlighted.

Several South American countries have a definition of a “waterway network” according to its economic capacity for the transport of goods or passengers:

- Brazil refers to the “economically viable” network, which includes inland waterways with cargo flows (existing or potential) exceeding 50,000 tons per year, highlighting the need

[2] The summary of the Colombian system is based on a technical report, prepared by Fernando Torres , TORGUN, in close consultations with the national experts in Colombia.

to develop a classification system for waterways also to improve the information available about its fluvial system.

- Peru presents a concept of the "main commercial waterway network", which includes rivers that are navigable during most of the year, which, given the almost total lack of roads, constitute the guiding axis of development, economy and fluvial integration of the Peruvian Amazon, as well as the only means of mass communication.
- As mentioned before, Colombia makes a distinction between waterways related to cargo transport (transportation of large volumes over long distances and oriented to exports, imports and commercial exchange among regions of the city system) and the transport of passengers and goods (connection between isolated towns and regions). In both categories, Colombia determines the appropriate routes for "major navigation" or "minor navigation".

There is an established practice in South America to guarantee on the main stretches of the most important waterways a minimum operational depth of the waterway. This is done both at the national level and on the international watercourses, such as Hidrovía Paraguay Paraná.

III. METHODOLOGY PROPOSAL FOR THE INLAND WATERWAY CLASSIFICATION FOR SOUTH AMERICA

Based on the elements described before, the current methodological proposal to be discussed at the Working Group Meeting on 8 June 2018, is based on the **following basic principles**:

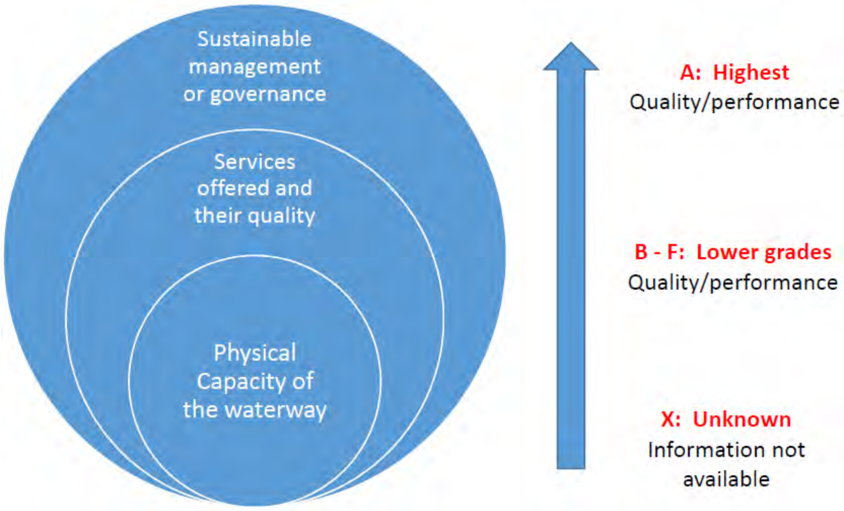
- For practical purposes, the current classification proposal will be restricted to the "shallow" waters, excluding deep-sea navigation, given the great difference in the parameters and exigencies of services, as well as the existing national and regional regulations dealing with the deep-sea navigation;
- The classification will be based both on the physical dimensions of the waterways and the level of services offered to the navigation. In this sense, the classification will go in line with the concept of the "infrastructure services", and will not be restricted to the purely physical characteristics of the infrastructure but will also account for the flow of services that a waterway provides.
- The classification will apply to both freight and passenger transport on inland waterways;
- The classification will include, in one form or another, some elements on the sustainability dimensions in the development and use of inland waterways.

In **its general structure**, it is proposed that the classification of inland waterways be based on the three sets of criteria (three-tier classification):

- Physical capacity of the waterway for the transport of cargo or passengers;
- The level of services to navigation available on the waterway
- The quality of the sustainable management of the waterway

Figure 1:

Basic structure for the inland waterways classification for South America



Source: ECLAC/PIANC Working Group on PIANC/InCom/ECLAC on Policies and Development of the Inland Waterway Classification for South America, 2018*.*

In each area, specific classification parameters shall define the class of the waterway. The classification will allow a partial application to account for the lack of data or updated information.

In Tier One, the primary parameter for the classification, according to capacity of the waterway, shall be the guaranteed minimum water depth of the navigation channel. The primary parameter shall be complemented by the division based on the limits to the values for the vessel's and convoy's length and beam during the period of low water conditions.

Table 4
Classes according to the capacity of the waterway

Class	Minimum operational depth (m)	Maximum beam of the vessel or convoy (m)	Maximum length of the vessel or convoy (m)
A	3,5	48*	280*
B	3,0	33*	210*
C	2,5	25*	120*
D	2,0	23*	140*
E	1,5	16*	80*
F	1,0	12*	50*
X	Data not available	Data not available	Data not available

Note: the value of the intervals shall be determined through analysis of the available fleet data.

In Tier 2: at the second level, the class (or the category) shall be determined by the quantity of specific services to navigation provided on the waterways, according to a matrix to be elaborated in consultations with other expert and technical groups, especially the RIS experts.

Table 5
Draft classification matrix according to the level of service.

	Continued Navigati	Continued Navig	RIS	AIS	Intermodal	Navigation signs	TBC	Navigational charts	Hydrographic surveys

	on 365/365 *	ation 24/7*			connections	and signals			
A	x	x	x	x	x	x	x	x	x
B		x	x	x	x	x	x	x	x
C			x	x	x	x	x	x	x
D				x	x	x	x	x	x
E					x	x	x	x	x
F						x	x	x	x
X	Data not available	Data not available	Data not available	Data not available	Data not available	Data not available	Data not available	Data not available	Data not available

This matrix will define the services included. For instance, continued navigability could be defined as navigability ensured throughout the navigation period with the exception of Breaks due to severe climatic conditions resulting in low water discharge or other impediments to navigation (for fixed periods that are kept to a minimum) and maintenance of locks and waterways (for fixed periods that are kept to a minimum). Navigation 24 hours/day will be defined as covering working days and reasonable hours on public holidays and weekends.

As an example, a stretch of the waterway with the maximum level of services shall be classified A. The waterway offering minimum level of services shall be classified F. The waterway missing RIS and AIS services and not offering continued navigation shall be classified: E. The waterways offering all services but RIS and AIS shall be classified [TBC].

Finally, **Tier Three** would include some parameters pertaining to the quality of the management shall be focused on the extent to which the waterway is managed in a sustainable manner.

Table 6
Parameters on sustainable management.

Class	Environmental sustainability	Social sustainability	Economic sustainability	Institutional sustainability
A	x	x	x	x
B		x	x	x
C			x	x
D				X
E	Two of the four.			
F	No management system in place			
X	Information not available			

By a way of examples, the parameters could include:

- Institutional sustainability: existence of the dedicated institutions in charge of the waterway's development, effective division of responsibilities and coordination mechanisms.
- Economic sustainability: existence of investment plans and financing schemes for the development of the waterway
- Social sustainability: existence of the rules and practices to deal with the social implications of the waterway development.
- Environmental sustainability: existence of the rules and practices to deal with the environmental implications of the waterway development.

The resulting classification would classify a waterway with maximum capacity, the maximum level of service and the best management system as classified: A-A-A. A waterway with maximum guaranteed depth but limitations in the beam and length of the vessels, the intermediate level of service and the limited management system would be classified: A (B-F)-D-E. Where information is not available, the classification can be B(X-X)-B-X.

IV. Pending issues and the next steps

The first classification proposal offers several advantages in line with the objectives and scope of the classification:

- The first two tiers of the classification, which captures both the physical capacity of the waterways and the level of services, offer both the elements needed to estimate the extent of the public or public infrastructure investments (to upgrade or maintain the level of services), and a minimum information on the operational requirements for the benefit of the private sector and other users of inland navigation. At the same time, the classification effectively delinks the physical capacity of the waterway from the level of service, allowing the smaller waterways to receive a separate higher rating if the quality of services is ensured;
- Tier Three, if operationalized, can pave a way towards incorporating sustainability and governance concerns and enhance the quality of the management of the waterways.
- The system offers a strong degree of flexibility, can accommodate additional criteria in all tiers and allows partial and gradual application. It also offers synergies and potential for the incorporation of the technical standards, such as RIS.
- The resulting full classification should identify the weakest links and elements in each waterway and in the network, as a whole.

At the same time, there are numerous pending issues to address before arriving at a functional classification methodology, including:

- As it stands now, the classification does not deal with the issues of availability and/or reliability and of how the minimum capacity is guaranteed in terms of the periods of time or percentage of navigation period or in terms of the waterway's coverage (navigable portion of the waterways (in width and length));
- The exact values for the intervals need to be defined based on a set of the representative values for South American inland navigation. For instance, the exact number of classes according to the dimensions of vessels should be defined based on the pragmatic analysis of the value added of shorter/larger intervals between dividing values from the perspective of shipping (practices and needs) and infrastructure works. The concept of "special conditions" could be used to offer a bit more of flexibility in using classes
- There is a need to clarify the key concepts used. The final document should include a glossary with the definition of the main terms used, such a class, a category. If possible, it should also contain guidelines, as to guide countries in its application and corresponding decision making on inland waterways infrastructure development and maintenance.

The current proposal will undergo several technical discussions to confirm the basic structure and the exact classification criteria and then a series of pilot applications to arrive at a final methodological proposal.

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A NEW SEA LOCK IN TERNEUZEN WITH EQUAL DIMENSIONS AS THE PANAMA CANAL EXPANSION PROJECT

Koen Van Doorslaer[1], Marc Bool[2], Wim De Cock[3], Harm Verbeek[4]

ABSTRACT

The construction has started for a New Lock in the lock complex of Terneuzen. By 2022 this lock, in the middle of a continuous operational lock complex, will be finished. With a lock chamber of 427m x 55m x NAP -16.44m, the New Lock in Terneuzen will have the same dimensions as the Panama Canal Expansion Project. This paper, a joint paper by Contractor and Client, describes the tender procedure and goes into detail on the project phasing and highlights the strong points of this project. By means of a temporary navigation channel around the construction area, the lock complex maintains its current three lock capacity for most of the construction period. The New Lock is built by a combined dry-wet construction methodology: building the contours from water, reclaiming the land in between, and continuing all civil works from land, this to keep the construction area as economical as possible. Due to the challenging geo-hydrology of the area, there is a risk of influencing the ground water level outside the project area and affecting the city of Terneuzen. This is solved by means of deep diaphragm walls up to a level of about NAP -45m for the lock head walls. Besides the New Lock, the project scope also requires new mooring and berthing facilities, deepening the inner and outer harbor and modifying the Flood Defense Line to guarantee safety against superstorms.

1. INTRODUCTION

The Rotterdam-Paris inland waterway route (Figure 1) is an important connection for the economy of France, Belgium, The Netherlands and other neighboring countries. This route however has some major bottlenecks. Most of them are or will be tackled in the Seine-Scheldt project, a joint plan between France, Wallonia and Flanders (the 2 Belgian regions). A last bottleneck is the Terneuzen lock complex in The Netherlands. This bottleneck and its renewed solution are the topic of the current publication.

Terneuzen, a city in the Netherlands with a little over 55.000 inhabitants, is housing a lock complex with three sea locks connecting the Ghent-Terneuzen Canal (BE) to the river “Westerschelde” (NL/BE). The Ghent-Terneuzen Canal is the main waterway to/from the Port of Ghent (BE) and is a part of the Rotterdam-Paris inland waterway route. Coming from this canal, the Terneuzen lock complex is the gate to the Westerschelde, the North Sea, the port of Antwerp, and gives via Hansweert also connection to the port of Rotterdam, see Figure 2.

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Figure 1. The Paris-Rotterdam Waterways, with the location of the New Lock Terneuzen

Figure 2. Ghent-Terneuzen Canal gives access to the Westerschelde, the North Sea, Rotterdam and Antwerp

The current lock complex is showing long waiting times for inland navigation and is since some time at its maximum capacity. Besides this discomfort, the existing locks in the lock complex are old and limited in dimensions:

- the Eastern Lock (1963) has dimensions of 280m x 24m, for inland waterway only;
- the Middle Lock (1910) is 140m long by 18m wide, for inland waterway and sea going vessels with a draught up to 7.2m;
- the Western Lock (1968) has dimensions 290m x 40m, for inland waterway and sea going vessels with a draught up to 12.5m;

Therefore, the Flemish (BE) and Dutch (NL) government have decided to replace the Middle Lock for a new and bigger sea lock in Terneuzen. With a lock chamber of 427m by 55m, the New Lock in Terneuzen will have the same size lock chamber as the Panama Canal Expansion Project that became operational in June 2016. Terneuzen's New Lock will be operational by mid-2022, after a construction period of 5 years. During all this time, the existing lock complex has to remain operational.

2. TIMELINE AND TENDER PROCEDURE

2.1 Timeline

The project New Lock Terneuzen is a binational Flemish (BE) - Dutch (BL) cooperation, both for the Client (VNSC, see section 2.5) and for the Contractor (Sassevaart, see section 2.6). In this section, an overview of the different phases is given starting from the first idea until the operational New Lock.

In 2006, first studies were started to evaluate the effect of the increasing traffic and ship sizes to the economy in the Ghent-Terneuzen Canal area. It showed that the lock complex in Terneuzen is a bottleneck and a hazard for economic growth. From 2007 on, cost-benefit analysis took place, proposing different solutions and investigating all of their possible aspects such as nautical safety, environment, traffic, stakeholders, …

On March 19 2012 the Flemish minister of Transportation and Public Works, and her Dutch colleague minister of Infrastructure and Environment, announced that a new Lock will be built in Terneuzen. The technical and functional requirements, the location of the lock, its geometry and other characteristics were not yet fixed at that moment, but should be the result of the plan study that both governments ordered in 2012. Between 2012 and 2015 this plan study took place, leading to two important documents:

- an EIA (environmental impact assessment) in which all different alternative solutions and their effect on different environmental aspects such as water, noise, air quality, nature, traffic and flood risk were studied.
- An OTB (Ontwerp Tracé Besluit, Dutch for “draft planning decision”), in which the choice between the different alternatives is made, thereby proposing a fixed location and geometry for the lock, the layout of roads and lock facilities. Also the legal framework for the project is given, which laws to obey, how to deal with topics such as air quality, traffic, archaeology, landscape, nuisance (noise, vibrations, …) for local residents, hinder for navigation (obstructions due to the works), …

Both the EIA and the OTB were published mid-2015 and open to the public for 6 weeks to file comments or complaints. After a period (+/- 1 year) of thorough investigation by the administrations of The Netherlands and Flanders, the comments and/or complaints led to the improved and official document, called “Tracébesluit” (Dutch for “final planning decision”), published on March 14 2016. This document gives a detailed description of the project, where the exact location of the lock is decided by both administrations. After again six weeks for comments or complaints by the public, a European tender was initiated on May 25 2016 for this Design, Construct and Maintain project.

2.2 Project details

The project that was put to the market is summarized in the following assignments:

- Design and construction of the New Lock with dimensions 427m x 55m x NAP - 16.44m within the existing lock complex of Terneuzen; NAP is the Dutch reference level.
- Realising the local operation and control of the New Lock and preparing for remote operation;
- The construction of four lock doors and two bascule bridges;
- Adapting the Flood Defence Line;

- The construction of a tugboat harbour and warehouse;
- Demolishing of the existing Middle Lock and houses on the lock complex;
- Adapting the lock infrastructure and terrains;
- Adapting the roadway infrastructure on the lock complex;
- Deepening the navigation channels and maintaining the nautical depth during the works;
- Maintenance for two years after commissioning, with the exclusion of the dredging works.

In Figure 3, the project area is shown. In transparent, the scope to be removed is shown. In black, the new constructions are shown. This project is more than building a New Lock. Also the dikes, mooring and berthing facilities for navigation and different quays are renewed within this project.

The Ghent-Terneuzen Canal has its still water level at +2.13m NAP, whereas the Scheldt River fluctuates between -1.89m NAP and 2.29m NAP in daily situation.

The New Lock will allow passage for large seaworthy vessels of 366m long, 49m wide and a draught of 14,5m.

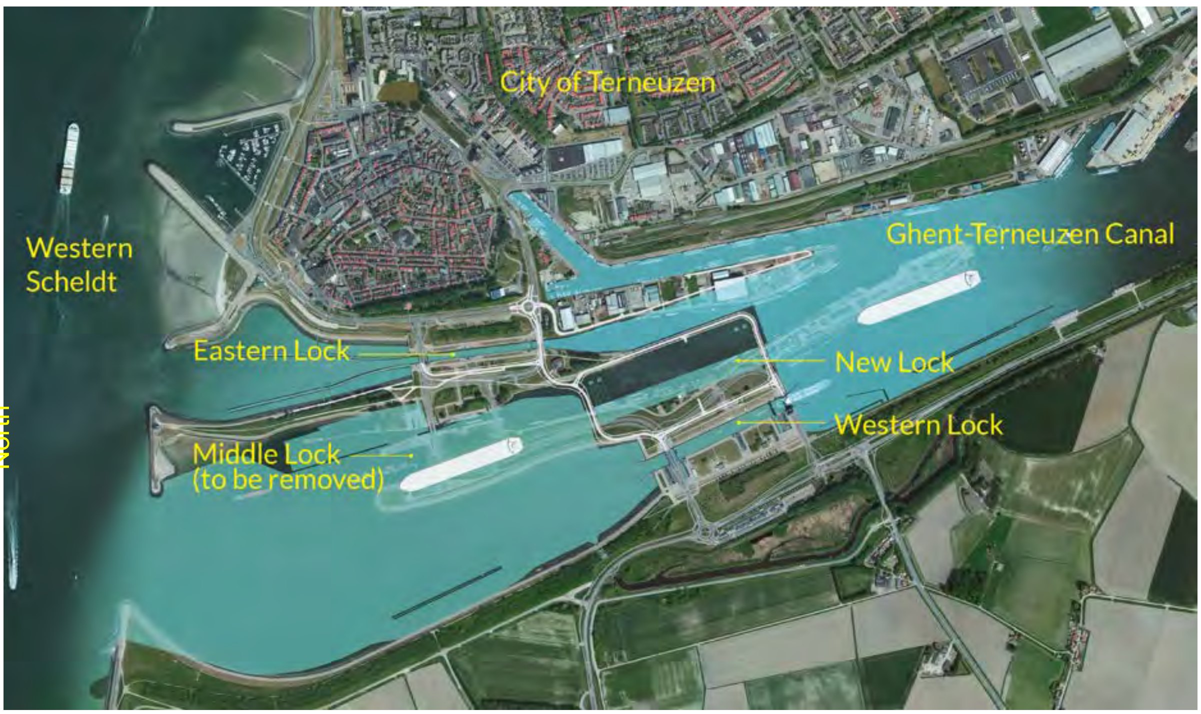

Figure 3. Project New Lock Terneuzen, with indication of the existing locks, the new constructions (black) and constructions to be removed (transparent)

2.3 Tender Procedure

The winning Contractor/Joint Venture (JV) is chosen based on the Dutch EMVI principle, where EMVI is the Dutch abbreviation for ‘Economical Most Valued Offer’. This means that besides pricing, also other criteria are valued in awarding the tender to the winning JV.

Just like any other tender, the tender started with a selection phase to select X participants by July 8 2017. Three Joint Ventures were selected for participation.

The 2nd step in the tender procedure was to go from X selected Contractors/joint ventures to a maximum of three by <u>November 2016</u>, by means of a risk management plan. Despite X was equal to three from the beginning, the selection procedure was maintained but no participants were excluded from continuing the tender procedures at the end of this step.

The 3rd phase of the tender was an individual dialogue phase between each JV and the Client, to discuss critical design scope and the EMVI-criteria. This dialogue finished end of March 2017, and the tender had to be submitted by all JV’s on <u>May 4 2017</u>.

After a review period by the Client and their advisors, the Project was awarded to the winning JV Sassevaart in September 2017.

The project will be finished by August 2022, after which a two year maintenance period of the New Lock and all new constructions with the exclusion of the maintenance dredging starts.

2.4 EMVI criteria

As stated earlier, the price was not the only criterion for selecting the winning JV. In The Netherlands this is known as an EMVI-procedure (Dutch: Economisch Meest Voordeling Inschrijving. English: Economical Most Valued Offer).

The first EMVI-criterion was to submit a Risk Management Plan (RMP) for 7 pre-selected risks:

- Insufficient knowledge and control of the subsoil. Design and construction methods are not aligned with the geotechnical and geo-hydrological situation of the subsoil (10 M€);
- Damage to the existing lock complex (40 M€);
- Delay on schedule (30 M€);
- Time delay and the experience of hinder for navigation (25 M€);
- Quality of the works, integrated design in combination with internal processes by the Contractor that are not mastered (25 M€);
- Imperfect connection in the existing lock complex; not or insufficient functioning of the industrial automated control of the New Lock (25 M€);
- Increased Life Cycle Cost (40 M€).

To mitigate these top 7 risks, a RMP had to be written of maximum 10 pages per risk. A score was given to every JV for each of his 7 risks (4 = excellent extra value, 3 = significant extra value, 2 = obvious extra value, 1 = extra value, 0 = no or little extra value), which then were converted to a (virtual) amount in Million euros. The maximal achievable (virtual) amount per risk is given in the list above between brackets, with a total of 195 M€.

The second EMVI-criterion was the possibility to bring a maximum of 2.9 million m³ sand (d_{50} > 150µm) as foreshore nourishment to Knokke-Heist (BE), for which a (virtual) amount of 3€/m³ was given to the JV who engaged to fulfil this nourishment. A total of 8.7 M€ could be earned by this second criterion.

The different biddings by the 3 JV's were compared to each other after subtracting the virtual earned amount of M€ from the offered price. In this manner, a JV with a higher price but a better EMVI-score could still win the tender. For example: a JV with a price of 600M€ and an EMVI-score of 100M€ (virtual total price for comparison 500M€), has a lower price but still loses the tender from a JV with a price of 620M€ and an EMVI-score of 150M€ (virtual total price for comparison 470M€).

The offered price had to be below 700M€ excl VAT. Together with the price, also the planning, a 4D model (BIM) and a landscape quality plan had to be submitted.

The winning JV will carry out the work for his offered price, not for the virtual price that was only for selecting the winning JV in the EMVI tender procedure.

2.5 Client

VNSC (Vlaams-Nederlandse Scheldecommissie, Dutch for Flemish-Dutch Commission for the river Scheldt) is a partnership between the Flemish (BE) and Dutch (NL) government to have a sustainable and balanced policy to manage the common Scheldt estuary. Topics like flooding, accessibility of the harbors and a healthy ecological system are dealt with by VNSC. And so is the New Lock in Terneuzen, that was agreed to be built by both governments in 2012.

VNSC set up a project board with Maritime Access Division of the Flemish Government and Rijkswaterstaat of the Dutch Government as members. Besides the project board, who are running the daily operations, there is also a steering committee with members of the project board and stakeholders like the Port of Ghent, the city Terneuzen, the province, …

The project is physically located in The Netherlands, and is thereby following Dutch Tender and Project procedures. The Netherlands are investing over 190M€ into this project, 48M€ is funded by the European CEF funds and the rest is paid by the Flemish government.

2.6 Contractor

The project was awarded to Joint Venture Sassevaart. This Flemish (BE) – Dutch (NL) combination exists of the following companies:

- DIMCO B.V. (DEME Infra Marine Contractor)
- Dredging International N.V.
- BAM Infra B.V.
- BAM Contractors N.V.
- Algemene Aannemingen Van Laere N.V.

Part of the DEME group

Part of BAM

Sassevaart has cooperation agreements with Engie (electrical and industrial automation) and Fugro (soil investigation). The design of the New Lock is done by the engineering departments of the 5 Sassevaart companies in corporation with Arcadis and IV-Infra.

Sassevaart will carry out the project for 626,6M€ (excl. VAT).

3. THE PROJECT

This section discusses some of the key elements that have contributed to JV Sassevaart winning the tender. But for a good understanding of the project, a first subsection briefly explains different phases of the project.

3.1 Project phasing

The project can be generalized in 6 major phases. Explanation is given below each figure. The navigation through the middle lock is always indicated by dotted lines, the water discharging (too high water level on the Canal is lowered by discharging water over the lock complex during low tide at the Westerschelde) by a twitched line with arrows and the Flood Defense Line (protection of the hinterland against storms) in red.

Figure 4. Phase 1: preparation phase

In Phase 1, the preparations of the project are carried out. Existing cables and wires are dug out and disconnected or relocated, houses, buildings, trees and other obstacles in the project area are removed. On the southern peninsula (indicated in blue in Figure 4) a logistic center is organized with a local concrete factory.

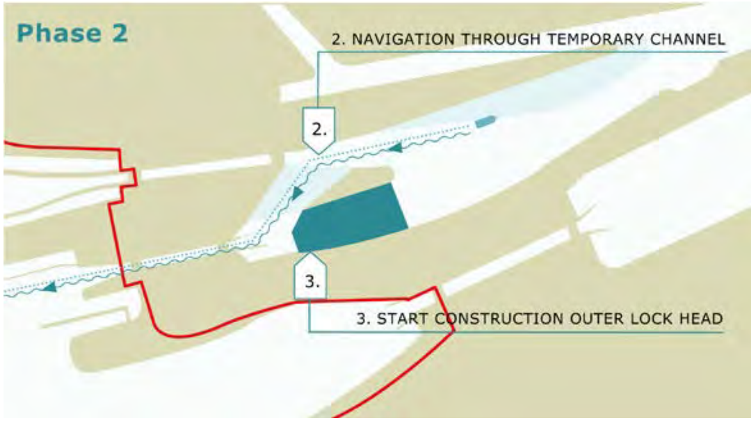

Figure 5. Phase 2: start land reclamation

Figure 5 shows Phase 2, where a temporary channel (see section 3.2) is dug and the land reclamation for the construction site is started. Traffic to/from the Middle Lock can still continue, but has to navigate around the construction area. To make this possible, also a part of the eastern terrain "Schependijk" is removed. The removed terrains are indicated in light blue and the new navigation way is shown by the dotted line.

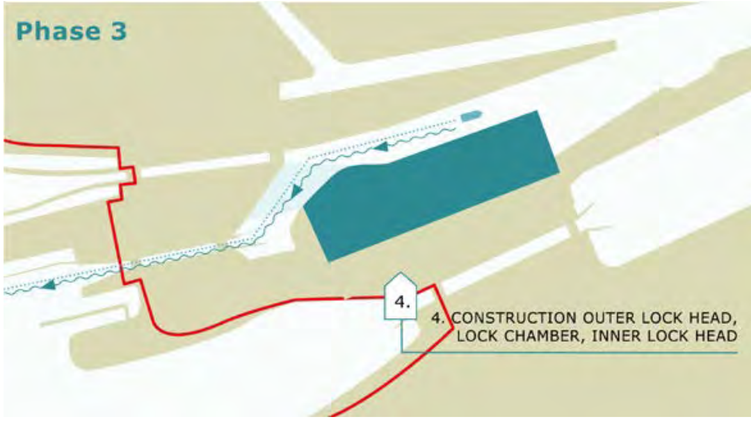

Figure 6. Phase 3: construction New Lock

In Phase 3 the land reclamation is completed and the construction of the New Lock is started. This phase is the longest phase of the project (duration over two years). The rest of outer lock head, the lock chamber and inner lock head are constructed in this phase. As shown on Figure 6, navigation goes similar to navigation in Phase 2.

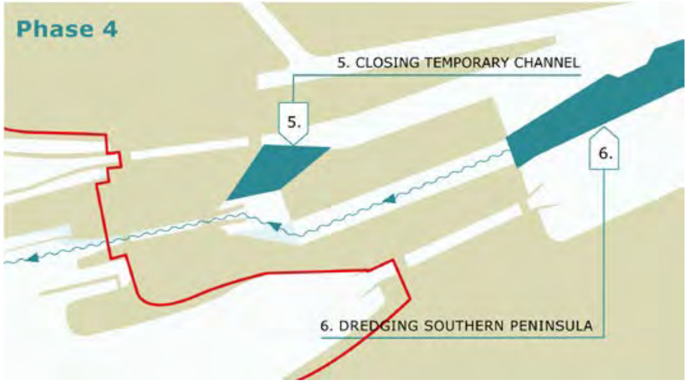

Figure 7. Phase 4: closing temporary cannel and Dredging the southern peninsula.

In Phase 4, the project is already reaching its last year. The civil works of the lock are being finalized and the logistic center can be removed. The southern peninsula, on the right in Figure 7, can thus be dredged and the lock doors, which will be constructed in China, can be installed. After the lock chamber is open, the temporary channel is closed for traffic (no more dotted line in Figure 7). The new lock is not yet open for navigation, the doors are in a testing phase but operate safely behind the original Flood Defense Line which is still in place (red line in Figure 7, equal to previous phases). During this testing phase of the doors, discharging water in times of high water on the Canal can be done through the lock chamber of the new lock that connects via open water to the filling and emptying system of the existing Middle Lock. This is indicated by the twitched line in Figure 7.

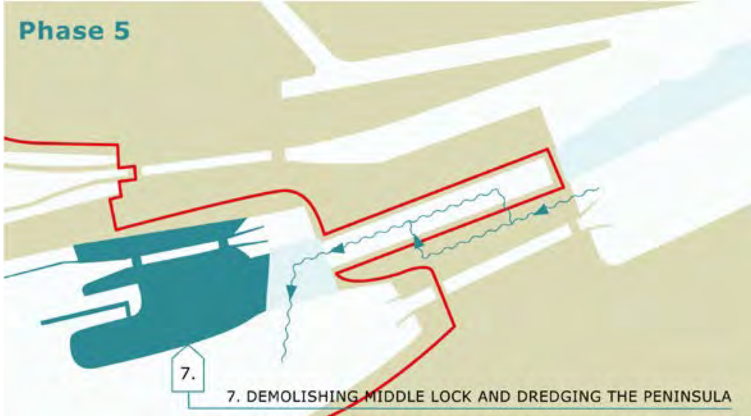

Figure 8. Phase 5: demolishing the middle lock and Dredging the northern peninsula

Phase 5 is for demolishing the existing Middle Lock and its surrounding northern peninsula as indicated in Figure 8. This phase can only start when the new Flood Defence Line, modified in Figure 8 compared to the previous phase, is constructed and offers safety against a storm with return period 4000 years. In this phase, discharging water in times of high water on the Canal is done by the filling and emptying system of the New Lock and by creating a discharging channel in this northern peninsula. This is again indicated by the twisted line in Figure 8.

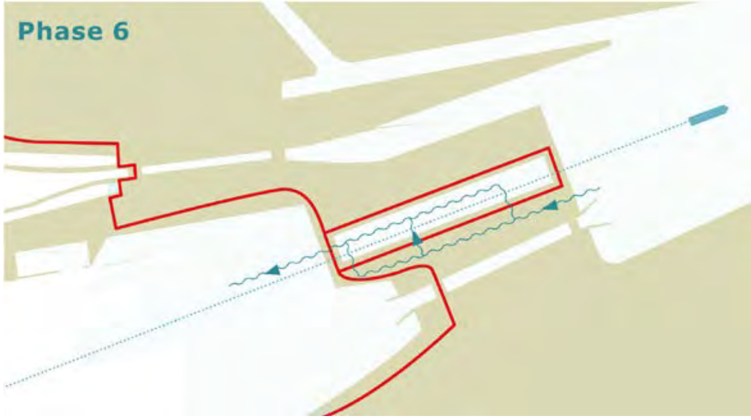

Figure 9. Phase 6: end of construction period, start of maintenance period

The project is finished in Phase 6, where the two year maintenance period starts. As mentioned earlier, the maintenance is not applicable for dredging works and maintenance of nautical depths.

For most of the construction period three locks are available for navigation. In Phase 1, 2 and 3 the three existing locks can be used. Then a short transition period of eleven (11) months in total with only two locks: Phases 4 and 5, where water levelling already uses the New Lock but no navigation is allowed yet. In Phase 6, the New Lock is fully operational offering a renewed lock complex with again three locks.

3.2 Temporary Channel

One of the Client's and stakeholders' main desires was to have a smooth construction period which will maximally avoid obstruction, time delays and other hinder for navigation. Industry along the Ghent-Terneuzen Canal and not in the least the Port of Ghent would benefit from a solid and sound solution that guarantees their daily operation and minimal delay for deliveries. Also shippers from inland navigation clearly stated their wishes to have a project solution that would minimize delay. For seaworthy vessels, VNSC already created a solution where a time-window is created for the seavessels to arrive in Terneuzen's lock complex and immediately be transferred through the Western, Middle or future New Lock. Since inland navigation has lower priority, they could become the victim of only two available locks during the five year lock construction period and increasing waiting times.

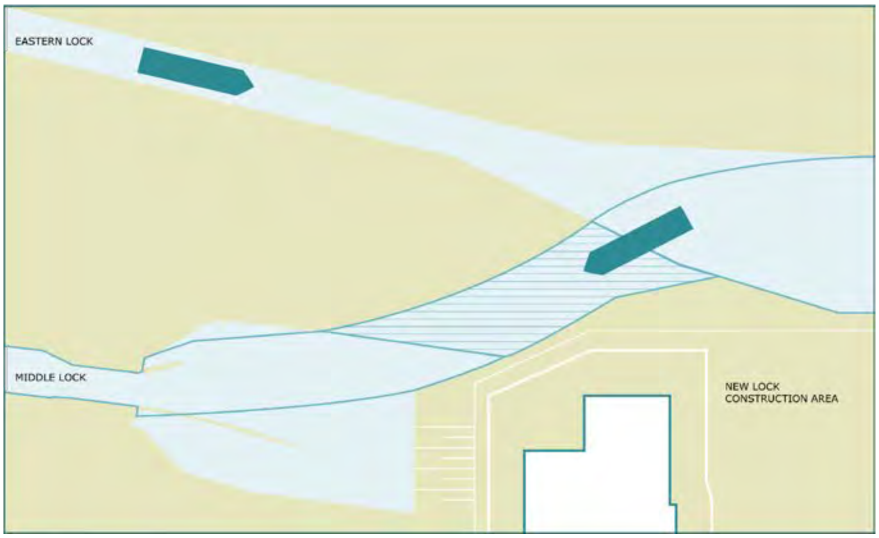

Figure 10. Navigation cannel created to/from the Middle Lock, going around the construction pit.

Sassevaart chose to maximally fulfill this desire of the inland vessel shippers and the Port of Ghent and organized an internal brainstorm session with consultants, shippers and design engineers to investigate different possibilities. The outcome was as sketched in Figure 10: digging a temporary navigation channel to create space for construction but at the same time keep the Middle Lock operational during most of the construction period. According to the Contract, the Middle Lock could be taken out of service upon start of the construction period thereby only having two operational locks. Due to this temporary navigation channel around the construction zone and by a smart planning, Sassevaart will have three locks operational for most of the time (two locks for a period of only eleven months of the five year construction period).

This solution was positively received by VNSC's project team, and two independent studies were carried out to study the feasibility and restrictions to navigation. The outcome was a navigation channel accessible for navigation up to class CEMT IVa or seaworthy ships of the same size (105m x 9.5m x max draught 3m). The temporary channel brings all traffic to/from the Middle Lock in the channel for the Eastern Lock, thereby influencing each other and not being able to transfer ships to the same side at the same time. These measures were taken into account, and the studies then showed a reduction in lost navigation hours by 50.000 to 75.000 and a 25% reduction of the passing time for inland navigation (compared to closing the Middle Lock). This measure has an economical value of 22 to 34 M€.

3.3 Reducing hinder for navigation

Besides the temporary channel, Sassevaart will provide other measures to reduce the hinder for shippers and navigation. Most of this is done by smart dredging and transport of dredged soil.

In the project there is a net volume of about 9 million m³ soil to be removed from the project. With siltation of the harbor during the 5 years construction period, creating working platforms for building the New Lock with soil from the project area and afterwards excavating the lock chambers and lock heads to their final depth, the brut volume soil to be handled is about 13 million m³. A large portion of this (about 40%) is located on the Canal side of the lock complex. A part of it is transported to the Canal side, but most of this soil will be deposited in the direction of the Westerschelde and Knokke-Heist and thus has to pass the lock complex. However, only 11% of the volume is transported by using the lock complex. The other and most significant portion is transported by means of floating pipes and land lines over the construction site (Figure 11) towards the outer harbor basin (Figure 12), where barges are filled to transport the dredged soil towards Westerschelde or Knokke-Heist. For the 11% soil that is transported via the Middle or Western Lock, extra barges are foreseen so the transport can happen mainly during the lee hours and avoid the busy hours of the lock complex. By foreseeing extra barges, full production dredging/excavating can be maintained to guarantee the planning. Finally, Sassevaart is not transporting soil through the lock complex during the eleven months that only two locks are available.


Figure 11. Floating pipe line and bridge construction of land lines over a road.

Figure 12. Cutter dredger (indicated location in orange) transporting its dredged soil via the green floating lines/land lines/bridges (Figure 11) to the barge (indicated location in purple) waiting in the outer harbor. The purple barge will sail direction of the Westerschelde/Knokke-Heist for disposal of dredged material.

Besides smart dredging and soil transport, a few other measures will be carried out by Sassevaart:

- More waiting places for ships during the phase that only two locks are available;
- A nautical advisor is fulltime available on the project;
- Periodic information sessions for shippers, stakeholders and people working on the lock complex;
- Deliveries to/from the logistic center happen in the lee hours of the lock complex.

The measures described in 3.2 and 3.3 lead to a maximum score on the EMVI-risk 4 “Time delay and the experience of hinder for navigation”. They give excellent extra value to the Contract’s requirements.

3.4 Construction technique

A lock can be built in different possible ways. The choice of construction method is always a very project specific choice. No two locks are the same, no two project conditions are the same.

A first possibility is a large open construction pit, like the latest and largest lock that was constructed in the Port of Antwerp (Belgium): Kieldrechtsluis (Figure 13). By creating a water retention wall around the construction site into the water-resisting clay layers, lowering the ground water level and excavating the soil inside the construction site, the lock can be built in dry conditions. This method often has more certainty, easy logistics, a good visual quality control, but uses more concrete (thick walls) and a lot of space. Since the New Lock partly has to come in the middle of an operational lock complex and partly on current waterways, an open construction pit wasn’t the most economic and smart space using solution.

Figure 13. Construction of Kieldrechtsluis in Antwerp by means of a dry construction method.

The opposite possibility is by building all civil constructions from the water with heavy cranes on floating pontoons, as shown in Figure 14. The use of space is smaller, the use of materials is much less, but it’s an expensive and time consuming method with lower workability, more difficult to guarantee quality and it might create hinder for navigation. Mainly this last risk was the reason for Sassevaart not to continue with this construction technique.

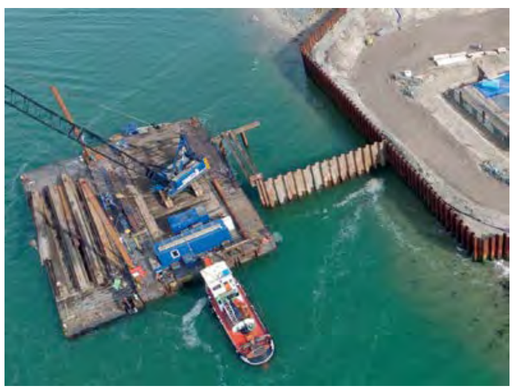

Figure 14. Example of wet construction method.

Besides these two extremes, many “hybrid” solutions are possible with partly wet, partly dry construction techniques. One example is IJmuiden (The Netherlands), where the largest lock in the world is currently being constructed also, like Terneuzen, in the middle of an operational lock complex. The construction zone in IJmuiden is built partly from land and partly from the water. First, combined walls are built along the contours of the new lock from the water side against the landside. When the construction zone was closed, land reclamation took place after which the lock chamber was built with deep diaphragm walls from the land (Figure 15). Simultaneously and also in a dry method, both lock heads are built and sunk to their final level by means of pressurized caissons. The lock chamber zone will then be dredged by a cutter suction dredger.

Figure 15. Construction of new lock in IJmuiden, Combined wet-dry construction method.

Different of the above mentioned and other possibilities have been investigated by Sassevaart for Terneuzen and a TOM (Trade Off Matrix) was set up to find the ideal solution with respect to cost, Contract, EMVI, safety, timing, environment, … The outcome was a combined wet and dry construction technique. The choice was also partly imposed by the geo-hydrologic from the underground, which will be explained in section 3.5. The construction technique by Sassevaart is presented below.

After the navigation channel is diverted as explained in Section 3.2, sheetpile walls are constructed from the water in the North, East and South end of the construction site and a combined wall is built from water on the western side. The eastern sheetpile quaywall is a permanent wall, which forms the eastern boundary of the construction zone during the works. The Eastern lock chamber wall, a combined wall, forms the western boundary of the land reclamation. The permanent walls are indicated in red in Figure 16. The other walls are temporary, for land reclamation purposes and are indicated in green in Figure 16. In between all four constructed walls, land reclamation takes place. These works are planned for end 2018-begin 2019.

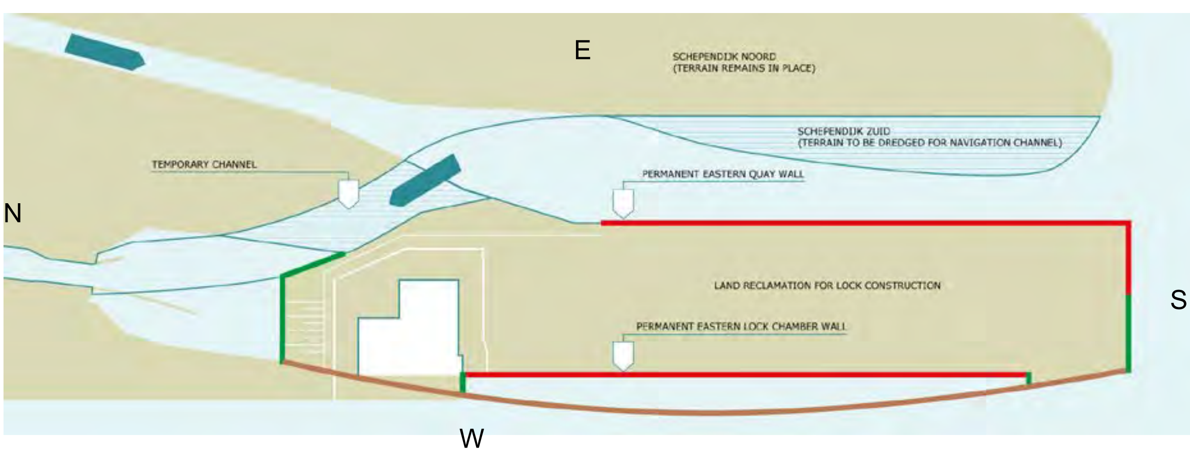

Figure 16. Land reclamation between 2 permanent walls (red) and 2 temporary walls (green)

On the dry (reclaimed) land in between the four boundaries of Figure 16, deep diaphragm walls are installed to create the lock heads, indicated in blue in Figure 17. Also the Western lock chamber wall is built from the (original) land with diaphragm walls (blue in Figure 16). This is the most elaborate construction phase (phase 3 of the project, section 3.1) which lasts until mid- 2021.

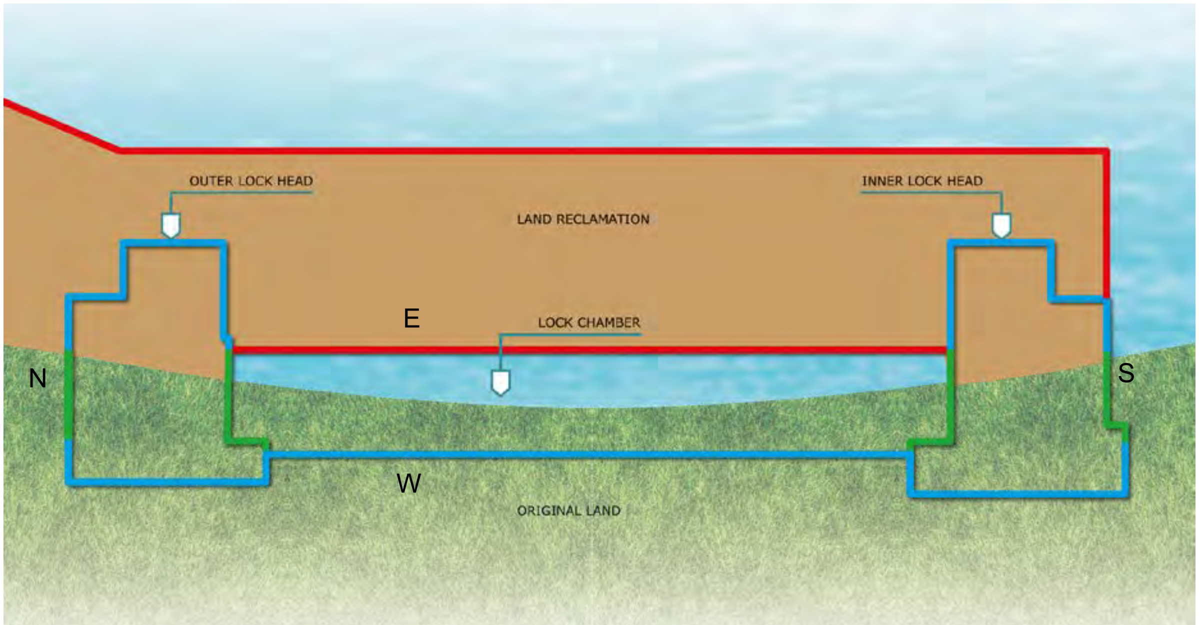

Figure 17. On the reclaimed land, deep diaphragm walls (blue) are built to form the lock heads and western lock chamber Wall.

The lock chamber, with its eastern wall a combined wall installed from water and its western wall a diaphragm wall installed from land, does principally not differ much in construction technique from IJmuiden: two walls are built, then excavated in between the two walls, finally under water concrete is installed as lock chamber floor. Unlike for Kieldrechtsluis (Figure 13), the lock chamber of New Lock Terneuzen is not designed to ever set dry.

The lock heads are built dry in a closed construction pit. To achieve this construction pit, diaphragm walls are built as contours. These diaphragm walls go to well below the water resisting clay layer ("Boomse Clay") and even have combined walls as reinforcement, see section 3.5. In between the diaphragm walls, partly dry excavation (up to NAP – 10m) is carried out, and for stability reasons the deeper excavations (up to NAP – 22m) have to be done with an excavator on a pontoon after the construction pit is filled with water up to NAP level again. After the excavation of the lock heads, a thick underwater concrete floor is installed which acts as strut, required for stability reasons. After this underwater floor is hardened, the water in the construction pit can be pumped out to continue the dry construction of the lock chamber.

3.5 Geo-hydrologic situation

The reason for the deep diaphragm walls of the lock heads, the reason for not going for a large open construction pit and the reason for building the lock chamber walls less deep and where possible from the water, can all be found in the subsoil in Terneuzen and its geo-hydrologic situation.

In that part of the low lands Belgium and The Netherlands, a clay liner is present, called Boomse Clay, at a varying depth. In Terneuzen this clay is present from about NAP – 20m to NAP – 35m. Boomse Clay has perfect water resisting capacity and is for example one of the main reasons why all locks in the port of Antwerp are built in an open construction pit with dewatering and open excavation: a water resisting wall into the Boomse Clay is built around the construction site, and outside this wall no influence of the lowered ground water table can be noticed. However in Terneuzen, thorough soil investigation has shown that this clay liner is not overall present. It's present at both ends, the lock heads, but absent in the middle where the lock chamber has to come. This is visualized in Figure 18.

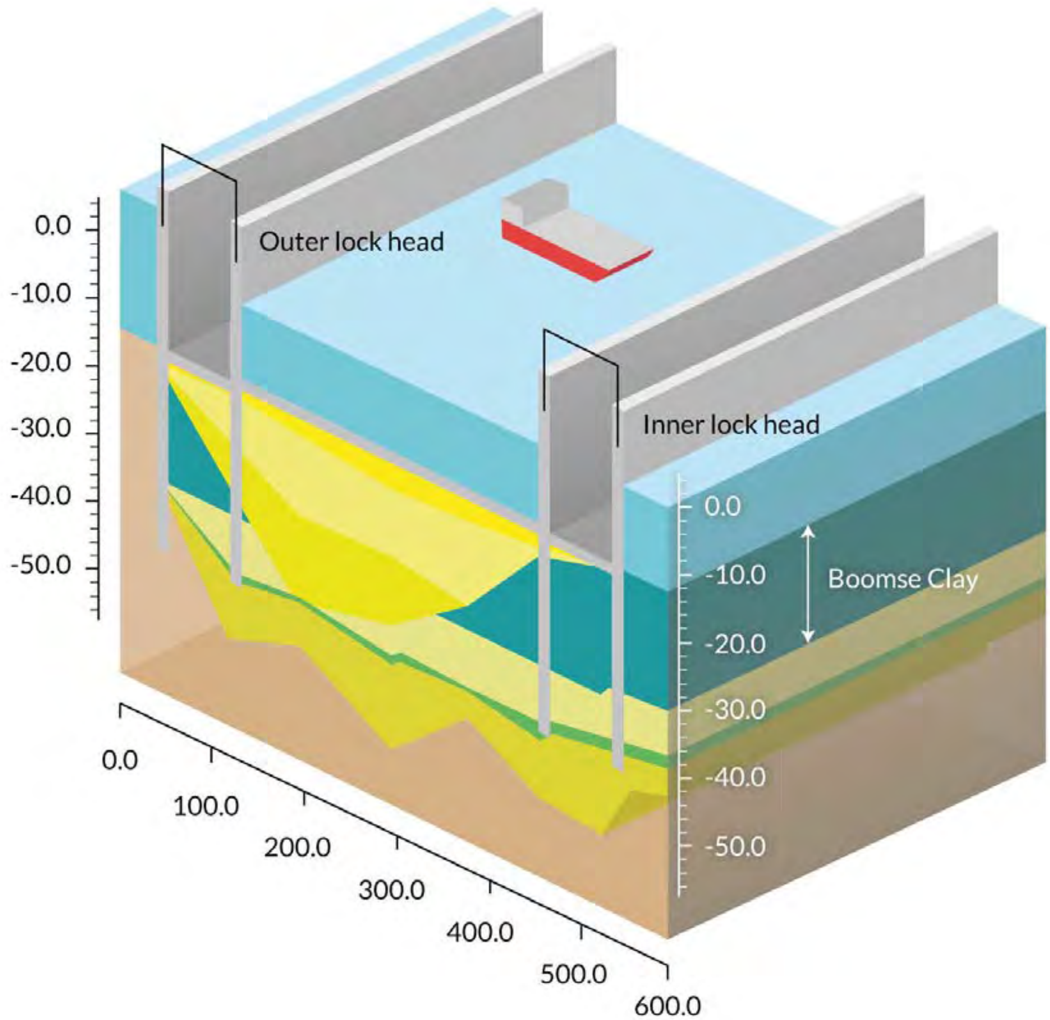

Figure 18. Geological Profile over the length axis of the lock, with indication of the Boomse clay.

Geological and historical investigations showed an ancient gully which has worn out the clay at this location. Through this gully, which got filled with fairly permeable sands, there is an open connection between the project area and the surroundings, such as Terneuzen city center. This has major consequences for the choice of construction technique. In the 1960's, when the Eastern and Western Locks were built, the Client and Contractors were less aware of this phenomenon and constructed the locks in open excavation with lowered water table. Due to this large extraction of ground water in the project area, also the ground water level in Terneuzen city lowered which lead to subsidence and damage to many houses. The church in Terneuzen even had to be broken down.

However, not dewatering at all would lead to high upward water pressures on the clay layer when the lock heads are fully excavated, and even risk uplift or bursting of the clay. Two possible solutions exist: installing pulling anchors or constructing (very) deep diaphragm walls.

- Drilling anchors through the concrete floor and anchoring them into the underground, would avoid the use of dewatering. Calculations showed a very high density of expensive anchors, making this an uneconomic and practically infeasible solution.
- Constructing (very) deep diaphragm walls as contours of the dewatering zone. Since the underground has much higher horizontal than vertical permeability it's important to block the horizontal water transport. Geohydrological calculations and a pumping test, to verify assumptions of soil layers' permeability, have shown that deep diaphragm walls until about NAP – 45m (10m below the Boomse Clay) are sufficient to allow a mild surface tension dewatering with returning

the water into the same soil layer. This has zero effect on the water table in Terneuzen city, which will intensively be monitored.

Creating a large open construction pit would require these very deep water resisting walls over large distances to contour the whole construction zone (which is already larger for an open excavation). This option was uneconomic and thereby not maintained by Sassevaart.

Constructing the lock chamber in the same way as the lock heads are foreseen, would also increase the length of (expensive) deep diaphragm walls. Therefore, only the lock heads will be carried out with deep diaphragm walls until about NAP – 45m. The lock chamber will be carried out as explained, by less deep combined (east) and diaphragm (west) walls with underwater concrete floor installation without ever setting the lock chamber dry.

3.6 More than just a lock

Apart from the New Lock all maritime mooring and berthing facilities are to be renewed by Sassevaart. This means deconstructing or adapting the present facilities and constructing new ones that are designed for the increased size of navigation that will pass through the New Lock. These new mooring and berthing structures are located at various locations within the Project area, shown in Figure 19.

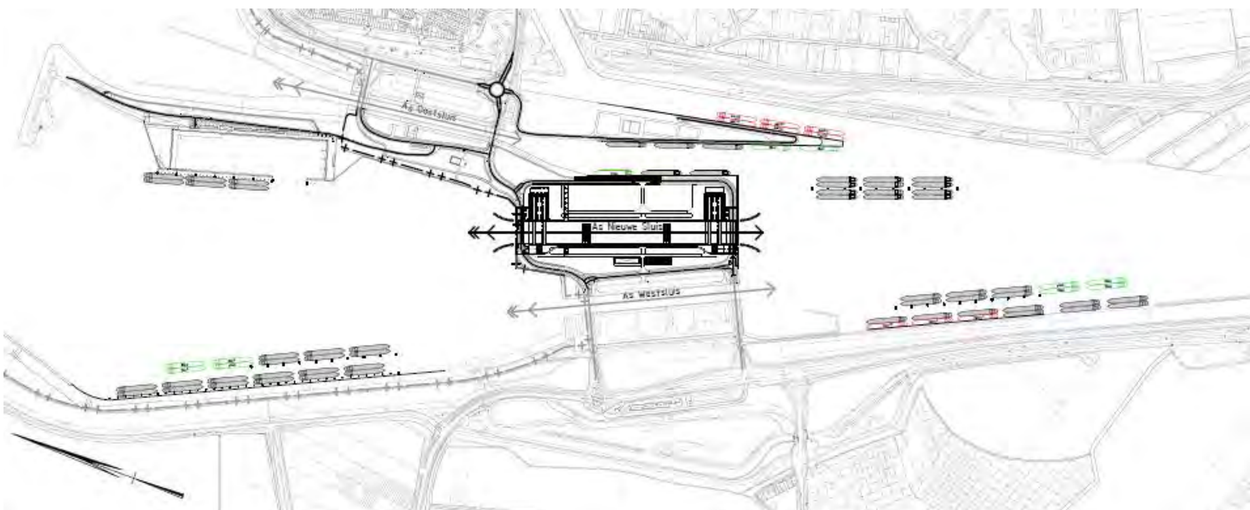

Figure 19. New Lock with new mooring and berthing facilities.

Since most of these facilities are located in and near the embankments of the inner and outer harbour, and also since these inner and outer harbours are to be deepened, the embankments itself are also to be adapted within this Project. The outer harbour embankments are part of the Flood Defence Line, so they have to be designed to withstand a storm with return period 4000 year.

The present Tugboat harbour, currently located on the northern peninsula, is also disappearing. Within the project, a new tug harbour (including various land- and offshore facilities) is to be constructed by Sassevaart.

Finally, also new buildings on the lock complex and new roadway infrastructure has to be designed, built and maintained for two years.

4. ACKNOWLEDGEMENTS

This paper is, like the project, a joint effort of many Belgian and Dutch colleagues, by the Contractor Sassevaart (represented by two authors) and the Client VNSC (represented by two authors). Besides the four authors, this paper and the whole project is a team performance by all people of Sassevaart and VNSC. A special word of thanks goes to the environment manager Nancy Vincke, the communication manager Lisette Booij and technical drawer Ankie Gijsel (Graphic Avenue) for their input and graphics.

NAUTICAL RISK ANALYSIS FOR VIDIN-CALAFAT BRIDGE IN THE DANUBE

by

Jose R. Iribarren [1], Raul Atienza[1], Jose I. Diez de Argote [2], Lourdes Pecharroman[1]

ABSTRACT

At the request of FCC Construccion, S.A. in connection with the Danube Bridge project between Vidin (Bulgaria) and Calafat (Romania), a Nautical Risk Analysis was developed to assess the probability of impact of vessels on the unprotected piles of the bridge located in the secondary channel of the river. The project consists of a combination viaduct - cable-stayed bridge taking advantage of the small island in the middle of the channel. In each of the piles of the main channel a circular fender is arranged. Piles in the secondary channel are not protected.

The objective of the study was to evaluate eventual risk of impact on unprotected piles, assessing the probability of occurrence and making a proposal for preventive or corrective measures. Complete traffic statistics were available, as well as information on the river flow, current velocities and wind, from which the actual navigation conditions in this section of the river were modeled.

The study analyzed the consequences of exceptional situations (breakdowns in propulsion or steering of vessels, lack of visibility, errors in position estimation, sudden bad weather, etc.), beyond the normal difficulties in sailing the river.

The analysis **identifies, analyzes and evaluates** different **expected risk situations** in the development of manoeuvres in order to establish effective preventive or corrective measures in accordance with the most stringent recommendations. Applying a ship manoeuvrability numerical model, the response of the vessels to each of the measures was analyzed according to the different possible situations.

Consequently, it was possible to evaluate the effectiveness of these countermeasures by determining, where appropriate, the "point of no return", which serves as a basis for the definition of restricted areas and development of contingency plans. During the development of the work a program of emergency simulations was established using model SHIPMA (Delft Hydraulics and MARIN (The Netherlands)). This model reproduces the behavior of a specific vessel during the execution of port access or exit maneuvers, subjected to the action of environmental agents (wind, current, waves, limited depth, shore suction, etc.) and aided by tugs, if any. For this purpose, it has an autopilot and tug control, which develop the necessary actions to maintain a desired track.

The working procedure was developed in the following phases:

- Identification of risk events
- Assessment of response manoeuvres
- Accidental risk assessment
- Definition of preventive and corrective measures

Bridges are outstanding structures in inland navigation. They are usually located and designed (including the fendering system) based on deterministic guidelines and standardized codes. Sometimes, the relevance of the bridge requires more detailed analysis in order to better identify operation conditions and optimize the structural design and the fendering system, taking into account risk of impact by vessels. This is especially important in areas with high density of traffic. Risk analysis is not the standard approach, but this paper shows an interesting case where it provided very useful information for design and permitted significant savings.

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1. INTRODUCTION

In response to the request from FCC Construcción, S.A. a Nautical Risk Analysis Study was developed in relation to the Danube Bridge Vidin-Calafat connecting Bulgaria and Romania. The aim of the study was to evaluate the risk of collision of vessels and barges with the bridge piles, pile caps and piers (hereinafter referred to as piers) located in the secondary branch of the river, such that an appropriate impact loading could be determined for use in the design of the vessel impact protection system for the piers in the non-navigation spans.

Figure 1: Bridge location

The bridge project consists of a combination viaduct plus tightened bridge taking advantage of the small island in the centre of the river. The piers of the bridge in the main river branch are protected against ship collisions by means of circular fender structures. The protection system for the piers of the viaduct in the secondary branch of the river was not developed at the time of the analysis.

Therefore, eventual risk scenarios related to vessel collisions with the viaduct piers should be evaluated. The probability of occurrence of such incidences was estimated and preventive or corrective measures were defined accordingly. An evaluation and justification of any potential for reducing the impact protection forces to be applied to the fixed structures (the piers) was undertaken.

The objective of the study was therefore the identification, analysis and evaluation of different risk hypotheses which could be expected during the development of the river transit. As a result, preventive or corrective actions and measures were defined and tested. As a reference, AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges (second edition 2009) and IMO (International Maritime Organization) recommendations were considered regarding risk situations. The study also took account of the reference included in the prEN 1991-1-7 Eurocode 1 - Actions on structures.

Two methods were applied in this study to determine the risk related to vessel collisions with the viaduct piers located in the secondary branch of the river:

- First method is a detailed risk analysis procedure for estimating the risk based on AASHTO Guide Specification.
- Second method is a cost-effectiveness analysis procedure for estimating the risk based on a simulation program covering emergency situations. A numerical model (SHIPMA) was used for the analysis.

The first method provides that the vessel collision requirements are aimed at preventing a vessel from impacting a bridge over a navigable waterway and causing excessive damage. A probabilistic model based on a worst-case-scenario, where a fully loaded fast moving vessel collides with a pier while moving unimpeded, is used to determine whether a bridge is adequately designed. In determining the feasibility of a given bridge it is necessary to consider the waterway geometry, the types of vessels using the waterway, the speed and load condition of the waterway vessels, and the response of the structure in the event of a vessel collision. If a structure is unable to resist the vessel collision forces, it needs to be protected by a fender system.

The second method is a cost-effectiveness analysis procedure for determining the risk level on a set of scenarios in which specific error situations or conditions are assumed to occur or exist on a ship prior to or during passage of the bridge. A simulation program covering emergency situations was defined and a numerical model was used for the analysis. This was SHIPMA, a fast-time manoeuvring model developed by Delft Hydraulics and MARIN (The Netherlands). This model computes the track and course angle of a vessel, taking into account the influence of wind, waves, currents, current gradient, variable water depth and bank suction. SHIPMA is a "fast-time" autopilot manoeuvring model, different from a real-time bridge simulator. Rudder, engine, bow thruster and tug control is done by a track-keeping autopilot that anticipates deviations from the desired track. A probabilistic model was also applied to obtain the annual frequency of collapse similar to that of the first method.

Figure 2: Bridge project Vidin-Calafat

The acceptable probability for any given bridge depends on the importance that the bridge serves to the community. Bridges may be categorized as either “critical” or “regular” according to AASHTO LRFD code Section 3.14.3. If a bridge is classified as critical, it must remain operational after a vessel collision. Once a bridge’s classification has been established, it is determined to have met the criteria according to its completed annual frequency of collapse.

The correct development of the risk analysis makes it necessary to manage detailed river traffic statistics (number and type/size of vessels upstream-downstream). A fluvial regime (distribution of water levels and corresponding current speeds) should be described as well, together with wind distribution (wind directions and speed). Based on the information available, different ship transit scenarios were defined in accordance with the existing conditions for navigation in this river section.

2. DATA COLLECTION

The first step in the vulnerability (risk) assessment process was to collect data concerning the waterway characteristics, the vessel fleet using the waterway, and the characteristics of the bridge.

The Danube Bridge Vidin - Calafat is located at km 796 of the Danube River crossing from the northern Bulgarian side to the southern Romanian side, which is due to a river bend that inverts the generally opposite location of the two countries. The Danube is the only major European river draining into south eastern direction into the Black Sea and with 2880 km length its second longest.

In the bridge location area the navigation route permits a 4400 m straight waterway. Under the bridge, the navigation width is narrowed to 150 m after reduction of the pier area. In the upstream and downstream part the route adapts from the wider bridge channel to the 200 metres total width of the navigation channel after 600 metres transition length.

Hydraulic characteristics were described by means of velocity distribution at the cross-sections for different elevations of the water surface. The vessel collision analysis was based on a water level of 32.0 meters and mean speed of the current of 1.2 m/s.

Climate conditions were analyzed by considering distribution of wind velocity from different directions for Vidin area. W sector collects wind data from SW, W and NW directions and E sector collects wind data from NE. E and SE are the most important sectors with a frequency of occurrence around 57.5% (210 days/year) and 31.7% (116 days/year), respectively.

Information about the number of vessel trips was obtained from ships that passed Iron Gates II lock located in Turnu Severing (Romania). This information was supplied by FCC Construcción, S.A. Manual analysis was executed to extract general vessel data and tonnage for the actual vessels transiting the waterway at the location of the bridge Vidin-Calafat. Additional information about the sizes and types of vessels was used from CEMT Class (European Conference of Ministers of Transport) classification.

The total number of estimated transits for the existing vessel design fleet is 4577 trips per year (about 13 trips per day). This total fleet consists of 1782 pusher convoys transits, 2033 self-propelled barge convoys and 371 passenger ships. Convoys are the predominant category of vessels navigating the Danube in this area.

PB1 to PB8 piers were considered for an impact of a vessel at level 36.0 m. The values of Ultimate Load State (ULS) or ultimate bridge resistance used for vessel impact analysis were calculated by FCC Construccion S.A. These values were used to calculate the probability of bridge collapse, computed according to the ratio of ultimate bridge resistance (ULS) to the vessel impact force.

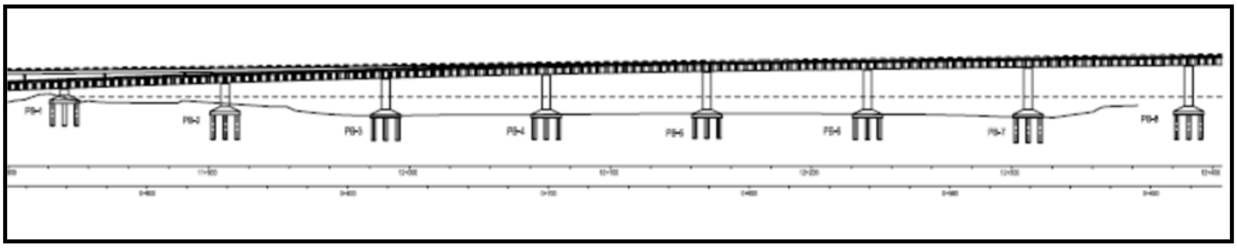

Figure 3: PB1 to PB8 piers

The data collection process is usually the most time-consuming part of the risk analysis. It has to be conducted in a thorough manner since the input values determined from the data collection will be used for the risk assessment. If the input data is incomplete or incorrect, the risk evaluation will be flawed and could result in incorrect conclusions. The data collected will come from a variety of disparate sources and it must be assessed and organized into a meaningful, coherent representation of the waterway, vessel fleet, and bridge characteristics.

3. AASHTO GUIDE SPECIFICATIONS. RISK ASSESSMENT

3.1. Introduction

This section describes the method to determine the risk related to vessel collisions with the viaduct piers located in the secondary branch of the river. It is a detailed analysis procedure for estimating the risk based on AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges.

In navigable waterway areas, where vessel collision by merchant ships and barges may be anticipated, bridge structures shall be designed to prevent collapse of the superstructure by considering the size and type of the vessel, available water depth, vessel speed and structure response. The AASHTO requirements apply to all bridge types which cross a navigable shallow draft inland waterway or canal with barge traffic, and deep draft waterways with large merchant ships.

The intent of the vessel collision AASHTO requirements is to establish analysis and design provisions to minimize bridge susceptibility to catastrophic collapse. The purpose of the provisions is to provide predictable design vessel collision effects in order to provide bridge components with a reasonable resistance to collapse.

Vidin-Calafat Bridge is over a navigable waterway meeting the criteria above. Therefore, it was evaluated as to vulnerability to vessel collision in order to determine prudent protective measures.

The AASHTO Guide Specifications contain three alternative analysis methods for determining the design vessel for each bridge component in the structure in accordance with two-tiered risk acceptance criteria:

- Method I is a simple to use semi-deterministic procedure
- Method II is a detailed risk analysis procedure
- Method III is a cost-effectiveness analysis of risk reduction procedure (based on a classical benefit/cost analysis)

The Guide Specifications require the use of Method II risk analysis for all bridges unless special circumstances exist as described in the code for the use of Method I and III. Special circumstances for using Method I include shallow draft waterways where the marine traffic consists almost exclusively of barges. Method III application includes very wide waterways with many piers exposed to collision, as well as existing bridges to be retrofitted.

Method II of the AASHTO Guide Specifications was applied. It is a probability-based risk analysis procedure for determining the appropriate vessel impact design loads for a bridge structure. Using Method II procedures, a mathematical risk model is used to estimate the annual frequency of bridge collapse based on the bridge pier/span geometry, ultimate resistance of the pier (or span), waterway characteristics, and the characteristics of the vessel fleet transiting the channel. The estimated risk of collapse is compared to standard acceptance criteria, and the bridge characteristics (span layout, pier strength, etc.) are adjusted until the acceptance criteria are satisfied. For example, if a structure is unable to resist the vessel collision forces, it needs to be protected with a fender system.

The acceptable probability for any given bridge depends on the importance that the bridge serves to the community. Bridges may be categorized as either “critical” (essential bridges) or “regular” (typical bridges) according to AASHTO Guide Specifications. If a bridge is classified as critical, it must remain operational after a vessel collision. Once a bridge’s classification has been established, it is determined to have met the criteria according to its completed annual frequency of collapse.

Vidin-Calafat bridge is classified as “critical”: the social/survival evaluation is largely concerned with the need for roadways connecting the communities located on opposite sides of the waterway.

Critical bridges is a situation where Method I should not generally be used. So Method II of the AASHTO Guide Specifications was applied.

The AASHTO Guide Specifications code uses annual frequency of collapse to determine whether a bridge design is satisfactory. An alternative way of representing a bridge’s vulnerability is with the inverse of annual frequency of collapse, or return period. A bridge’s return period is the number of years on average that a bridge may be expected to stand before a vessel collides with it and causes it to collapse. The annual frequency of collapse resulting from collision of a single pier by a vessel is calculated as follows:

$$AF_{ij} = (N_i)(PA_{ij})(PG_{ij})(PC_{ij})$$

where:

AFij = Annual frequency of collapse of pier j caused by vessel type i

Ni = Annual number of vessel type i (a vessel must pass all piers)

PAij = Probability of aberrancy of vessel type i with respect to pier j

PGij = Geometric probability associated with vessel type i and pier j

PCij = Probability of collapse of pier j due to vessel type i

The equation suggests that the annual frequency of collapse is based on a number of different probabilities. In sequence we need to know the probability that a vessel becomes aberrant; then, the probability that a vessel will strike the bridge given that it becomes aberrant; and finally, the probability that the bridge will collapse given that a vessel is aberrant and strikes the bridge.

The overall annual frequency of collapse of a bridge, AFTotal, is the sum of the annual frequencies that result from collisions of the various vessel types with the various bridge piers that one deemed vulnerable due to their location relative to the channel. Thus, we have:

$$AF_{Total} = \sum_{i=1}^{NV}\sum_{j=1}^{NP} AF_{ij}$$

where:

AFTotal = Annual frequency of collapse of the bridge

NV = Number of vessel types (i.e., including the same loading condition, size, etc.) that pass the bridge

NP = Number of bridge piers within three times the overall length (LOA) of the vessel from the navigable channel centreline

The sequence of computations is such that the annual frequency of collapse is determined for each pier and the sum of these frequencies for all piers provides the overall annual frequency of collapse of the bridge. For a bridge classified as “critical,” the annual frequency of collapse must not be higher than 0.0001, which means that its return period must not be shorter than 10000 years. The required annual frequency of collapse for a bridge designated as “regular” must be no larger than 0.001 corresponding to a return period of 1000 years. In terms of these acceptable levels, we have:

$$AF_{Total} < AF_{Acp}$$

where:

AFTotal = Annual frequency of collapse of the bridge

AFAcp = Acceptable annual frequency of collapse of the bridge

3.2. Acceptance Criteria

Risk can be defined as the potential realization of unwanted consequences of an event. Both a probability of occurrence of an event and the magnitude of its consequences are involved. Risk estimation is the process used for controlling such risk and arriving at an acceptable level of risk.

There are many approaches to evaluating risks in order to determine acceptability. The most important of these can be grouped into two broad categories:

- risk comparison approaches
- cost-effectiveness of risk reduction

In this study, risk comparison was used to establish the Method II acceptance criteria. The AASHTO Guide Specifications established an acceptance criterion of frequency of collapse, AF, for bridge collapse associated with vessel collision:

AF < 0.0001 per year for critical/essential bridges

AF < 0.001 per year for typical bridges

The acceptable annual frequency of bridge collapse for the total bridge as determined above shall be distributed over the number of pier and span elements located within the waterway, or within the distance 3 x LOA on each side of the inbound and outbound vessel transit paths if the waterway is wide. This results in an acceptable risk criterion for each pier and span element of the total bridge.

Vidin-Calafat bridge is classified as “critical”, as the social/survival evaluation is largely concerned with the need for roadways connecting the communities located on opposite sides of the waterway, so it should comply with the criterion AF < 0.0001 per year.

3.3. Probability of Aberrancy (PA)

The probability of aberrancy is the likelihood that a vessel deviates off course due to pilot error, poor weather conditions or mechanical failure. One of the three main components to determine the annual frequency of bridge collapse, the probability of aberrancy can be calculated by two different methods. The first method involves performing a statistical analysis of historical data from a given channel. While this method is the most accurate, it can be time consuming and difficult. The simplified approach detailed in AASHTO LRFD 3.14.5.2.3 is an approximation method and can be written:

$$PA = (BR)(R_B)(R_C)(R_{XC})(R_D)$$

where:

PA = Probability of aberrancy

BR = Aberrancy base rate

RB = Correction factor for bridge location

RC = Correction factor for current acting parallel to vessel transit path

RXC = Correction factor for cross-current acting perpendicular to vessel transit path

RD = Correction factor for vessel traffic density

3.4. Geometric Probability (PG)

Once a vessel has become aberrant, it is then necessary to estimate the probability that the vessel will strike the bridge. To do this, geometric considerations are necessary. The geometric probability is based on a number of parameters including the geometry of the waterway, water depth, location of bridge piers, span clearance, sailing path of vessel, manoeuvring characteristics of the vessel, location, heading and velocity of vessel, rudder angle at time of failure, environmental conditions, width, length and shape of vessel, and vessel draft.

3.5. Probability of Collapse (PC)

Given that a vessel has gone aberrant and has struck a pier, it is then necessary to estimate the probability that the bridge will collapse. Several variables including vessel size, type, configuration, speed, direction of impact and mass influence the probability of collapse. The stiffness of the bridge piers and the nature of bridge superstructure also influence the probability of bridge collapse. The AASHTO LRFD code Section 3.14.5.4 which addresses probability of collapse was developed by Cowiconsult (1987) based on studies performed by Fujii and Shiobara (1978) using Japanese historical damage data on vessels colliding at sea (AASHTO LRFD C3.14.5.4). The ratio of ultimate lateral resistance to the vessel impact force is computed in order to estimate the probability of collapse.

3.6. Risk analysis

Based on the risk analysis results for the non-navigation piers of Vidin-Calafat Bridge, 8 piers were exposed to potential collision (although piers PB-1 and PB-2 would be exposed only if very high water levels occurred).

The annual frequency of bridge collapse estimated was AF = 0.0000225 (a corresponding return period of 1 collapse in 44389 years).

The acceptance criteria for a critical/essential bridge classification is an annual frequency of bridge collapse equal to, or less than, AF = 0.0001 (a return period of 1 in 10000 years).

Therefore, the annual frequency of bridge collapse was lower than the acceptance criteria for a critical/essential bridge, so regarding the protection system for the piers of the viaduct in the secondary branch of the river, PB1 to PB8 located in the non-navigation zone, they did not require to be protected against ship collisions.

3.7. Sensitivity Analysis

A sensitivity analysis was executed for the most influential parameters in the risk assessment developed in previous sections:

- Vmax - design impact speed
- Vmin - minimum design impact speed
- Displacement - Length
- Aberrancy Base Rate
- Increase in number of Vessel Trips

In all cases analyzed the value of the annual frequency of collapse was lower than the acceptance criteria. High values of Vmax -design impact speed- ≥5.6 m/s for downbound transit and ≥4.2 m/s for upbound transit resulted in corresponding values of annual frequency of collapse 0.000114 > 0.0001 (acceptance criteria). These values of navigation speed are not permitted in the area, because the ferry service Vidin-Calafat requires the ships to reduce speed and increase the vigilance.

4. SHIP NAVIGATION NUMERICAL MODEL (SHIPMA). RISK ASSESSMENT

4.1. Introduction

This section describes the second method to determine the risk related to vessel collisions with the viaduct piers located in the secondary branch of the river. It is a cost-effectiveness analysis procedure to determine the risk level on a set of scenarios in which specific error situations or conditions are assumed to occur or exist on a ship prior to or during passage of the bridge.

A simulation program covering emergency situations was defined and a ship manoeuvring numerical model was used for the analysis. This is SHIPMA, a fast-time manoeuvring model developed by Delft Hydraulics and MARIN (The Netherlands). This model computes the track and course angle of a vessel, taking into account the influence of wind, waves, currents, current gradient, variable water depth and bank suction. SHIPMA is a "fast-time" autopilot manoeuvring model, different from a real-time bridge simulator. Rudder, engine, bow thruster and tug control is done by a track-keeping autopilot that anticipates deviations from the desired track. A probabilistic model was applied to estimate the annual frequency of collapse comparable to the first method.

4.2. Description of SHIPMA model

The study was developed applying the fast-time ship manoeuvring model SHIPMA, developed by Delft Hydraulics-MARIN (The Netherlands). This program simulates the manoeuvring behaviour of a ship.

The mathematical model computes the track and course angle of the vessel, taking into account the influences of wind, waves, currents, shallow water and bank suction. Rudder, engine and tug control is effectuated by a track-keeping auto-pilot that anticipates deviations from desired track and changes in currents. Control algorithms are also available for bow and stern thrusters.

The application of SHIPMA is primarily in port and fairway design, referring to both approach channels and inland waterways. According to PIANC a first estimate of the required channel width based on their methodology has to be followed by ship manoeuvring simulations. The mathematical model is used in port design and inland waterway studies to give the designer an insight into the inherent possibilities and restrictions of vessels, infrastructure, and environmental conditions. Based on the insights gained, mitigation, if needed, of the infrastructure design (channel layout, manoeuvring basin and terminal layout) and/or the admittance policy can be proposed. In a more detailed design the SHIPMA study will usually be followed by a study with a real-time manoeuvring simulator.

The following flow diagram gives an overview of the program structure:

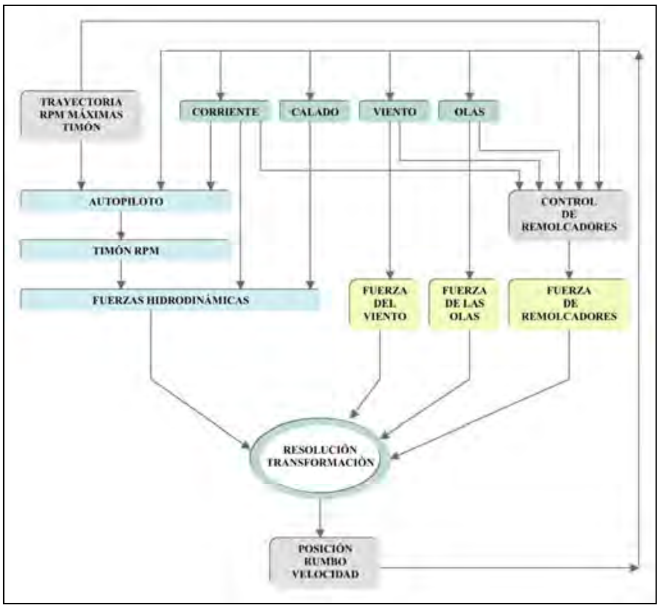

Figure 4: Flow diagram of SHIPMA model

4.3. Hazard Identification

All accident scenarios with significant contributions to the risk of collision are identified and analyzed in this phase. A very large fraction (about 80%) of navigational accidents are caused by human error or negligence, and detailed representation of human error is therefore necessary for the methodology to be successful.

An alternative approach in ship collision risk analysis is to base the risk of collisions on a set of scenarios in which specific error situations or conditions are assumed to occur or exist on a ship prior to or during passage of the bridge.

Emergency manoeuvres were simulated with the selected vessels using SHIPMA model in order to identify those cases in which the vessels drift towards the piers of the bridge when they are either sailing upstream or downstream in the Danube river.

The following emergency manoeuvres were considered:

- Loss of propulsion (black-out): at a specific point when passing near the bridge the engines fail and the vessel begins to drift under the influence of the prevailing conditions (wind and current).
- Extreme weather conditions (wind): in the vicinity of the bridge the wind suddenly increases. As a result, the vessel is sailing under extreme conditions.
- Escape manoeuvre: required following some type of error (poor visibility, incorrect positioning, etc.) which modifies the ship course towards the non-navigable branch of the river. The ship's rudder is then turned hard to avoid entering the risk area. This manoeuvre was carried out with the rudder hard to port and starboard.
- Crash-stop: required following some type of error (poor visibility, incorrect positioning, etc.) which modifies the ship course toward the non-navigable branch of the river. Eventually engines are put full astern to stop the ship and avoid entering the risk area.

This study only analyzed the engine shut down sailing the Danube because it was considered the main reason that could make the vessel drift toward the bridge piers. According to this, black-out manoeuvres were simulated.

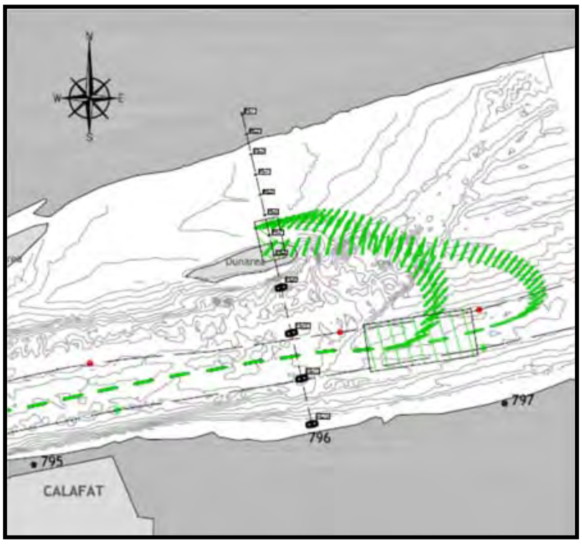

Figure 5: Black-out manoeuvres upstream

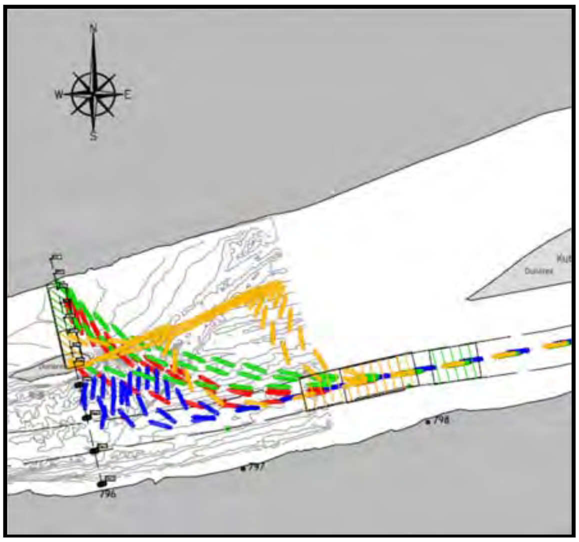

Figure 6: Black-out manoeuvres downstream

4.4. Description of the Vessels

Four inland waterway vessels typically used in the area were analyzed:

- Class IV (ES) (82.9x9.5x2.5 m. Displacement 1600t)
- Class Vb (SV) (187.0x11.4x2.8 m. Displacement 4924 t)
- Class VIb (SV) (185.0x22.8x3.0 m. Displacement 10102 t)
- Passenger Ship (130.0x22.0x1.7 m. Displacement 4000 t)

4.5. Assessment of Manoeuvres

For each vessel, possible collision incidents with the piers (PB-1 to PB-8) of the bridge in Danube river were analyzed due to loss of propulsion of the vessel.

The cases studied considered that the vessel sailed upstream or downstream affected by wind from different directions, under the effect of river flow. Three possible locations for loss of engine-power were used as benchmarks for each condition, so that a precaution area was established, associated with loss of propulsion or any other circumstance that caused the ship to drift toward the piers. Therefore, a sensitive navigation area was defined affecting bridge piers safety.

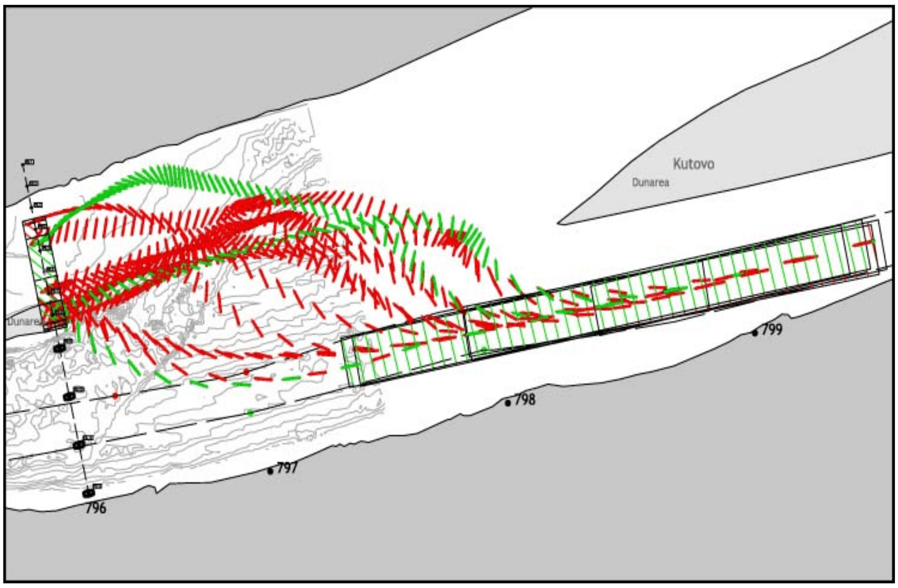

Figure 7: Black-out manoeuvres downstream and sensitive area

4.6. Frequency Analysis

The collision frequency analysis was carried out by identifying critical situations, denoted collision scenarios, where a ship-bridge collision could occur. A model was then developed for each scenario and an estimate of the frequency of collision as a result of the specific critical situation was computed. Information regarding waterway and bridge layout, weather conditions and ship traffic was used as input to the model to yield an estimate of the annual ship-bridge collision frequency.

The collision frequency computations took the following scenarios into account:

- "Powered Collisions": collision occurs when a vessel is directly on collision course towards a pier and no evasive actions are carried out. This collision type is also referred to as a collision due to human error.
- Drifting ship collision can occur when a vessel suffers an engine failure and drifts towards a pier.

4.7. Powered collision

The powered collision occurs when a ship, due to human failure, continues its course along the shipping routes directly into a pier. There are two assumptions which have to be satisfied in order for a powered collision to occur:

The vessel has to be on collision course, i.e. the direction of travel is directly towards a pier. Such vessels will be referred to as a "collision candidates"

A collision candidate must hold its course and not perform any evasive actions.

- The probability of sustaining a collision course is denoted "the probability of human error".
- The causes for human error can be the following:
  - Absence from bridge
  - Present but distracted
  - Present but incapacitated due to accident or illness
  - Present but asleep from fatigue
  - Present but incapacitated from alcohol
  - Ineffective radar use (bad visibility only)

The collision frequency for powered collisions on navigation path is calculated using the following equation:

$$F_C = \sum_{i=1}^{Ntr} N \cdot P_t(x) \cdot P_i(x) \cdot P_C \cdot P_{reac}(x)$$

where:

FC = Frequency of a passing vessel colliding under bridge (per year)

N = Total traffic on the type of vessel (vessels/year)

x = Position on the navigation path

Pt(x) = Probability of being on position x on the navigation path

Pi(x) = Poffset x Pcourse, probability of having a certain offset on the current x-position and probability of following a certain course heading towards the piers (course deviation)

PC = Probability of human failure or technical failure of navigational equipment, covering human error causes listed above

Preac(x) = Probability of the crew onboard being unable to react in time to correct the navigational error (depends on x)

4.8. Drifting ship collision

In case of failure in the propulsion engine the ship will start to drift, which will introduce a risk of collision if the drifting direction is towards the bridge pier. If a vessel is to collide with a pier, then the following conditions must be satisfied:

- The vessel has to be on collision course, i.e. the wind/current is moving the ship directly towards a pier. Such vessels will again be referred to as “collision candidates”
- A collision candidate must hold its course and not perform any evasive actions until the point of impact. The evasive actions considered are fixing the propulsion engine or successful anchoring.

For drifting collisions, a basic method is used to estimate the probability that ships experiencing a breakdown (i.e. loss of power, propulsion and/or steering) drift to a bridge pier. The model includes estimations of frequency of ship breakdown at specific locations and also of effectiveness of mechanisms that help take control of the vessel again. These mechanisms include emergency salvage, self repair, and anchoring.

The frequency of vessels drifting from a navigation path and colliding with an object near the lane can be written as:

$$F_{CD} = \sum_i N_i \bullet F_{drift} \bullet T_i \bullet P_{D1} \bullet P_{D2} \bullet P_{D3}$$

where

FCD = Collision frequency of drifting vessels (per year)

Ni = Number of vessels of ship type i in the area around the object (vessels/year)

Fdrift = Frequency of breakdown (per hour)

Ti = Average time a vessel of ship type i spends in the area to be considered for the calculation of the collision frequency (hours)

PD1 = Probability of the ship drifting towards the object

PD2 = Probability of not receiving any effective external help before a collision occurs

PD3 = Probability of no collision avoidance by the ship before a collision occurs (i.e. the crew is unable to stop the drifting through self-repair, anchoring, etc.)

If there are several navigation paths, the total collision frequency is the sum of the collision frequencies of each path.

4.9. Risk Analysis

In the previous section the expected frequency of ship collision with a pier in the bridge was estimated. In order to obtain the risk (frequency x consequences), the consequences must be determined.

In the event of a ship colliding with a pier, it could either lead to a severe damage of the bridge and/or the vessel. It may also be the case that the ship barely touches the bridge resulting in a less severe or insignificant damage.

In the following paragraphs, only consequences related to the collapse of the bridge are considered. Therefore, the probability of bridge collapse (PC) will be calculated due to a collision with an aberrant vessel (frequency of collision both powered and drifting, FC).

AF will be computed for each bridge pier and vessel classification:

$$AF = FC \times PC \quad = \text{risk}$$

The summation of all element AFs equals the annual frequency of collapse for the entire bridge.

Given that a vessel has gone aberrant and has struck a pier, it is then necessary to estimate the probability that the bridge will collapse. Several variables including vessel size, type, configuration, speed, direction of impact and mass influence the probability of collapse. The stiffness of the bridge pier and the nature of bridge superstructure also influence the probability of bridge collapse. The AASHTO LRFD code Section 3.14.5.4, which addresses probability of collapse was developed by Cowiconsult (1987) based on studies performed by Fujii and Shiobara (1978) using Japanese historical damage data on vessels colliding at sea (AASHTO LRFD C3.14.5.4). The ratio of ultimate lateral resistance to the vessel impact force is computed in order to estimate the probability of collapse (see section 3.4.3).

The following table summarizes the results obtained:

Table 1. Frequency of collision and annual frequency of collapse

Type of collision	Collision Frequency	Annual Freq. Collapse
Powered	1.41E-04	1.09E-05
Drifting	2.98E-03	3.85E-06
Total	**3.12E-03**	**1.48E-05**

5. CONCLUSIONS

The following table shows a comparison of the results obtained by the two methods applied:

Table 2. AASHTO Method and Numerical Model (SHIPMA). Summary of results

Method	Collision Frequency	Annual Freq. Collapse
AASHTO	1.17E-02	2.25E-05
NUMERIC MODEL	3.12E-03	1.48E-05
Method	**Collision Period**	**Collapse Period**
AASHTO	85	44389
NUMERIC MODEL	320	67796

Finally, the acceptance criteria recommended by AASHTO was applied (AF < 0.0001). For both methods, the annual frequencies of bridge collapse are lower than the acceptance criteria for a critical/essential bridge. Therefore, regarding the protection system for the piers of the viaduct in the secondary branch of the river, PB1 to PB8 located in the non-navigation zone, they do not require to be protected against ship collisions.

It is important to remember that the risk is a composite of two values: frequency of occurrence of a ship-bridge collision and its consequences. In this case, the expected frequencies of collision are 1 in 85 years according to AASHTO Method and 1 in 320 years according to Numerical Model (SHIPMA) results. These expected periods of collision could be interpreted in the sense that for a design life of 100 years, the probability of a ship-bridge collision is not negligible. However, the results of the computation of the annual frequency of collapse (return periods higher than 10000 years (acceptance criteria)) mean that if there would be a ship collision, the bridge piers will resist the impact and therefore the bridge will not collapse or the probability of collapse is extremely low.

The importance of the bridge pier impact resistance values should also be considered (PB-3 to PB-8) for the final result of the analysis using AASHTO methodology. According to this aspect, the value of the Ultimate Load State (ULS) for piers PB-3 to PB-8 used in calculations must be at least those considered in this study. So these values shall be the minimum protection required for the piers.

Bridges are outstanding structures in inland navigation. They are usually located and designed (including the fendering system) based on deterministic guidelines and standardized codes. Sometimes, the relevance of the bridge requires more detailed analysis in order to better identify operation conditions and optimize the structural design and the fendering system, taking into account risk of impact by vessels. This is especially important in areas with high density of traffic. Risk analysis is not the standard approach, but this paper shows an interesting case where it provided very useful information for design and permitted significant savings.

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5th Lock at Brunsbüttel

by

Dr. Matthias Schäfers[1] and Frank Allgäuer[2]

1. GENERAL

The 5th lock at Brunsbüttel (abbreviated as: 5SKB) is located at the entry from the North Sea to Kiel Canal. The Canal's traffic significance with reference to the Baltic Sea Region and the North Sea ports is great (Brunsbüttel, August 2016). It is the busiest artificial waterway in the world according to the number of ship movements. Due to the traffic flows from the North Sea ports to the Baltic Sea, the lock has great economic importance.

At present, the Brunsbüttel lock system has 4 lock chambers. Initially, two locks with mitre gates with a width of 24m, each, were constructed; however, they were soon supplemented by two more locks with sliding gates with a width of 46 m (42 m useful width), each. Theses locks have not been generally refurbished since their construction (1914). Due to their intense use and the high need for maintenance and repair, it is not possible to refurbish the existing locks during normal operation. For this reason, another lock has to be provided for the period of general refurbishment to maintain the existing capacities. This necessitates the construction of a 5th lock chamber. After the general refurbishment of the existing locks, the lock capacity will be larger than at present. The dimensions of the new lock were selected congruently with the two large existing locks.

Apart from the lock function, the structure is also very challenging as regards flood protection as well as Canal drainage. The structure is characterised by these two, partially opposing, but also supplementary requirements.

In consideration of the required lock availability and the comparatively large number of parallel locks with the same dimensions, the response time could be selected so that it is not necessary to keep a second gate permanently ready for operation at each lock head. This will reduce investment cost. Only must storage area be provided for the selected number of spare gates. In addition, the construction of a dry dock is currently being planned. Redundancy requirements comparable to the requirements of the gates rendering a lock chamber failure improbable are therefore made for the drive systems and the upper carriages.

Due to the variety of requirements made, large sea locks are very complex structures. The core of these systems and of even higher complexity are the lock gates and the ballasting/emptying system that will significantly affect the overall function of the lock in the future. These locks, which are designed with an integrated ballasting system in this case, are objects characterised by structural engineering, mechanical engineering and shipbuilding.

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Brunsbüttel is located on the mouth of the Elbe river to the North Sea, on the western end of Kiel Canal. This area of the Elbe river is seriously subject to tides involving significant silt loads. The lock is situated on a lock island at the entry of Kiel Canal; space is very confined on this island, and the location of the lock and especially the transverse extension of all components, including the drive units, had to be optimised.

Figure 1-1: Overview Brunsbüttel (source: Google Maps)

In this article, only a very limited number of challenges to be taken account of in connection with this type of lock gates can be addressed.

2. DESIGN SELECTION

The selected gates types for the 5SKB are sliding gates, designed so that they may also be used in the existing chambers of the large lock after the latter's general refurbishment. Therefore, the wheel-barrow system as used in the existing locks was selected. An undercarriage and a upper carriage on which the sliding gate is supported has been designed. Consequently, only an upper carriage and an undercarriage with the same connection system and a rail system as provided for the new lock have to be made available on the old lock after refurbishment, and the gates can easily be used in the old locks. This allows to waive spare gates and maintenance may be standardised. The investment cost for additional gate chambers and drive systems can hence be saved.

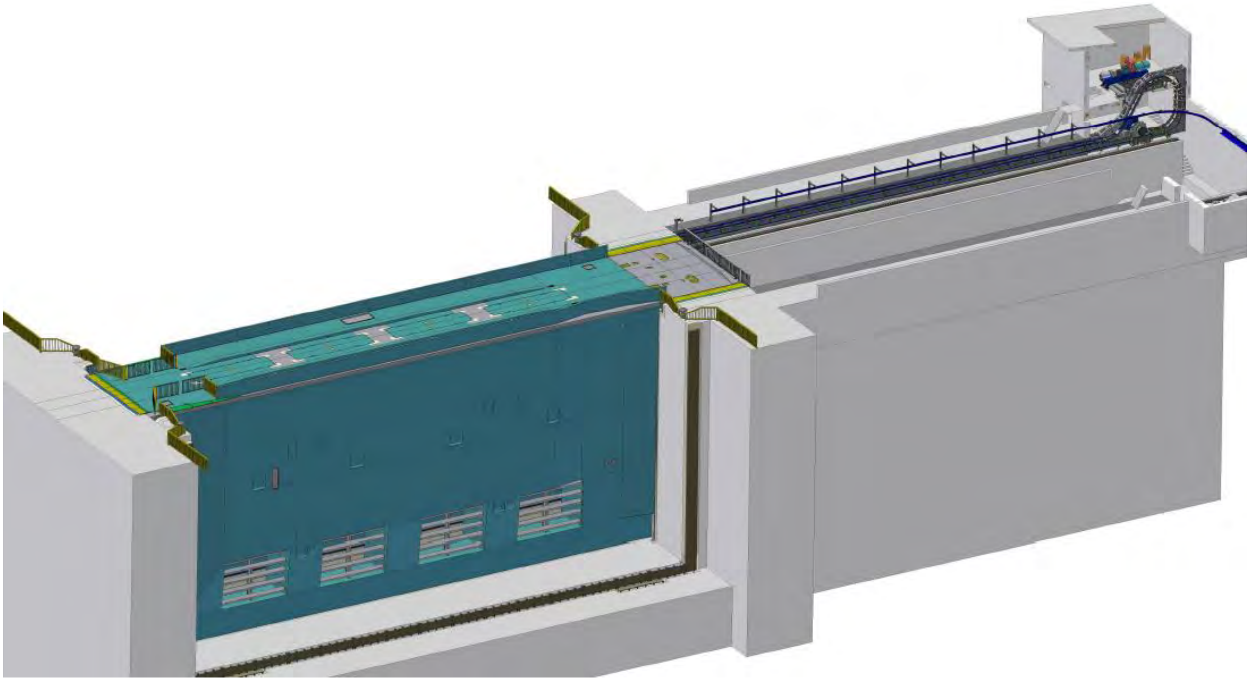

Figure 2-1 Presentation of sliding gate in closed position

The gates of the 5SKB were designed as floating structures; one purpose of this is to allow for a facilitated replacement of the sliding gates. For assembly/disassembly, the gates are floated by means of the above-mentioned ballasting/emptying system. General conditions for the assembly (this also applies to the disassembly) of the gates is an appropriate elevation of the bottom edge of the floating gate so that it may be swivelled over the undercarriage (function of ballasting and tide level) and a rotating swivel movement around the gate chamber may be initiated. In consideration of manufacturing tolerances in steel and concrete construction, a permissible heeling and a permissible trim, a few centimetres of space are sufficient to float in the gates.

The gates are solely assembled by means of tugboats as described above, so that they may be moved at low cost without the need to use specialised cranes. The spare gate may remain on dolphins until its next use. For general refurbishment, the use of a dry dock is convenient, since it provides more space than the gate chambers, and all necessary equipment is available to significantly facilitate maintenance and repair. The gate may easily be transferred between the locations by means of tugboats. Due to the mobility of the gates, a second gate chamber can be omitted.

As a result of all requirements of the gates, the latest generation of gates is heavier than in the past. In contrast to previous gates, the new gates are fitted with filling and emptying valves that increase the gates' weight in comparison to the old gates at Brunsbüttel lock. By providing the filling valves in the gate, maintenance can be performed completely independent from lock operation (e.g. in a dry dock).

3. STRESS

The gate is subject to a variety of impacts, which are generally compiled in accordance with DIN 19704. Especially the dead weight, the outside water pressure, ballast water, but also traffic loads on the roadway are to be noted. In the present section, only some particularities of stress for the sliding gate are addressed.

3.1. Fatigue

It is intended that the 5SKB manages 17 double locking processes each day. The term double locking process is illustrated in Figure 3-1.

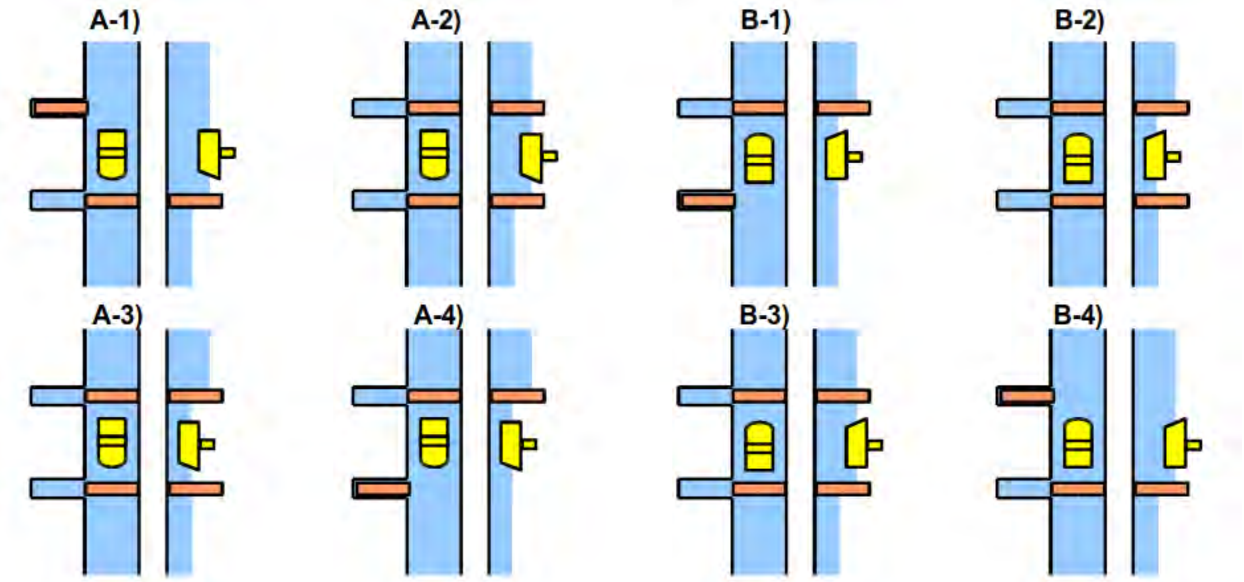

Figure 3-1: Definition of double locking process (A - locking from Kiel Canal to Elbe; B - locking from Elbe to Kiel Canal)

The North Sea is a sea characterised by intense tides, Kiel Canal has a constant water level of approx. ± 0.00 m asl. The North Sea's regular tidal range is ± 3.00 m. The gate is moved at almost constant water levels in the Canal and varying water levels on the North Sea. The water level in the lock chamber changes between that on the North Sea and that in Kiel Canal with each locking process. The outside head is always subject to the water pressure as it develops in relation to time with the tides. In Kiel Canal, the water level is largely constant, and in the lock chamber, the water level changes between these two levels with each locking process. The gates can only be opened and closed when the water level is almost balanced.

Figure 3-2 shows the tidal sequence over a day. The tidal sequence was approximated by a sine wave, which is sufficiently accurate for the investigation purposes. The tidal sequence is assumed congruently with a day. This will have almost no effects on the investigation result, but this simplification slightly renders results conservative.

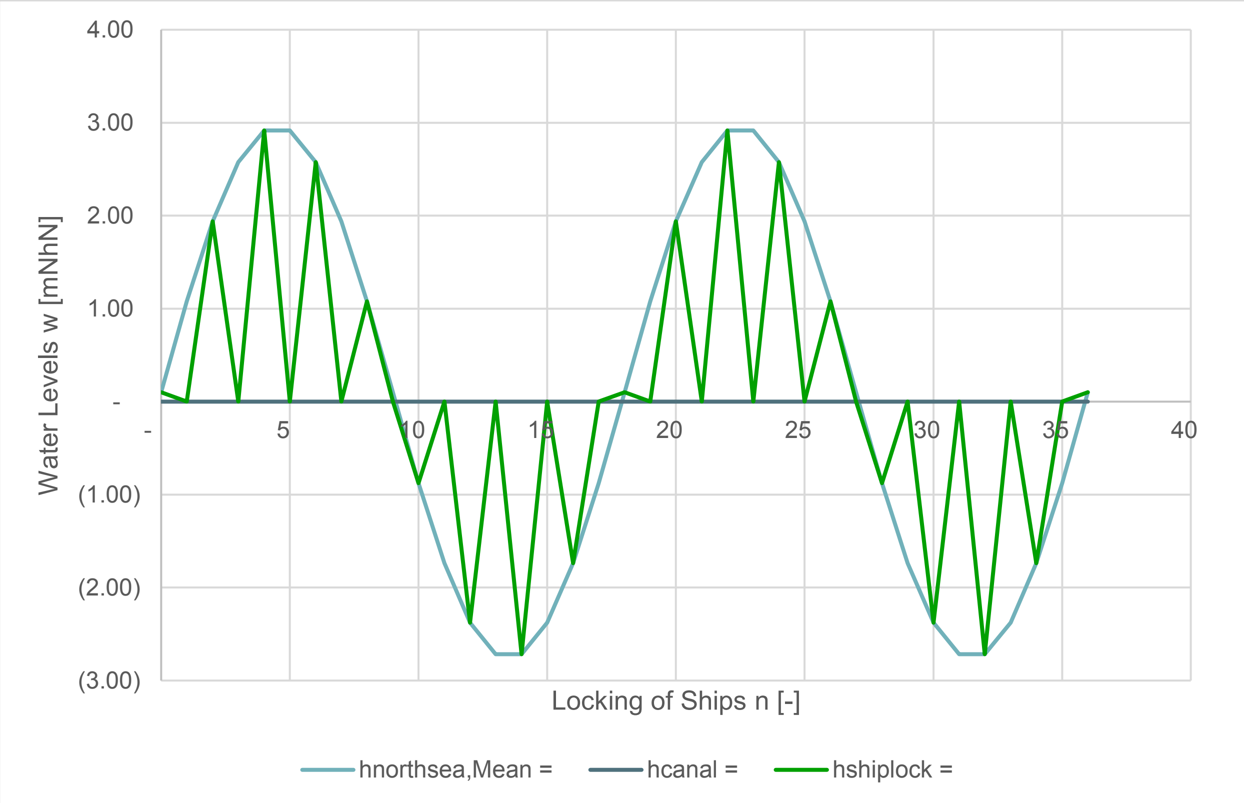

Figure 3-2: Illustration of tidal sequence over a day (17 double locking processes)

The progress of the water level differences acting horizontally on the gate is affine to the tidal sequence. This results in a fatigue stress on the gates, composed by the daily tidal sequence and the varying load conditions arising from gate opening and closing. Assuming the maximum load difference as the fatigue stress produces uneconomical results. It is also uneconomical to compute all water level differences individually, evaluate them at all relevant locations and optimise them at the design stage. Such procedure would significantly extend the design process. The mean value, however, would be on the insecure side, since high stress produces disproportional damage according to Wöhler's conception. To compute a reduced damage-equivalent, however conservative water level as input value for a FE computation, the damage hypothesis may be evaluated according to Palmgreen-Miner (DIN-EN-1993-1-9:, 2005).

$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} \qquad (1)$$

The given load changes n_i result from the locking frequency. The locking processes at the 5SKB are evenly distributed over the day, largely independent from the tidal range. There is hence is no effect from the fact that locking only takes place at high water levels, for instance. This results in a typical number of load cycles for each water head. The manageable load cycles N_i result from the existing notch types and the actual stresses for the given notch types according to the Wöhler curve.

$$D = \sum_{i=0}^{k} \frac{n_i}{N_{Rd,i}} = \frac{n_1}{N_{Rd,0}} + \frac{n_2}{N_{Rd,1}} + \cdots = \frac{k \cdot n_d}{N_{Rd,Ä}} \qquad (2)$$

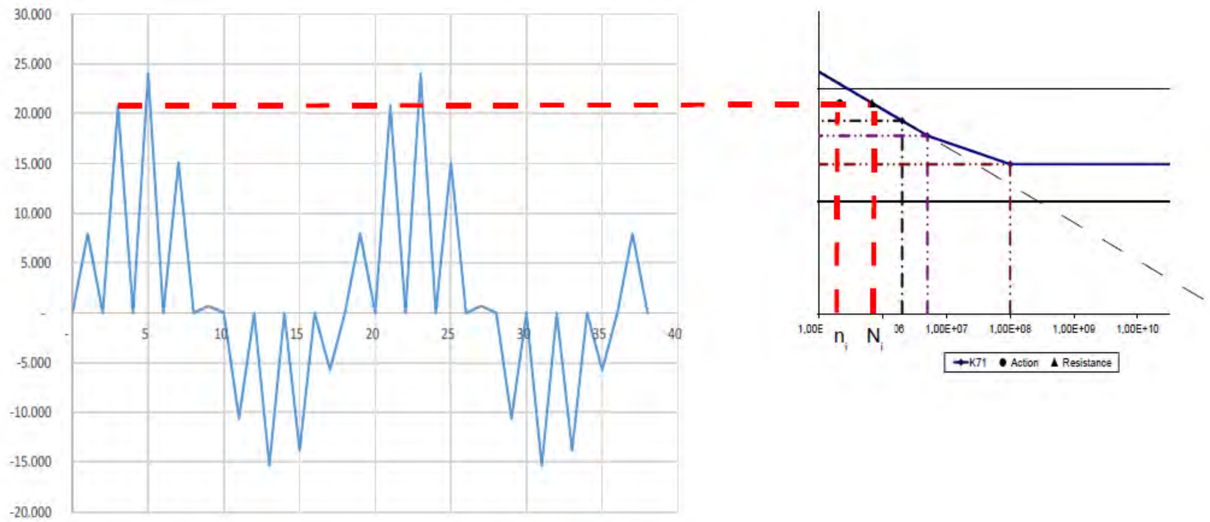

Figure 3-3 Schematic comparison of water level differences to Wöhler curves

The exact recording of fatigue stresses at all specific notches in combination with ductility matching the hydromechanical steel structure, may help to avoid cracks in the structures (in particular cracks due to fatigue and brittle fracture).

3.2. Maximum stress by waves

Another function of the sliding gate is flood protection. The lock is an integral part of the flood protection line along the North Sea coast. Storm tides may reach a height of approx. + 8.88 m asl. according to a forecast. The rated flood level is + 7.00 m asl. The water level determination and the resulting skinplate increase (to + 7.60 m asl.) takes account of the effects of the climate change.

The top gate area is solely loaded by waves in consideration of the expected rise in sea levels. During a flood, water for instance entering through leakage may also enter Kiel Canal. The amount of water entering by waves will not result in a significant rise of the Kiel Canal water level. For this reason, the sealing requirements have been reduced in these local areas where only waves act. ,

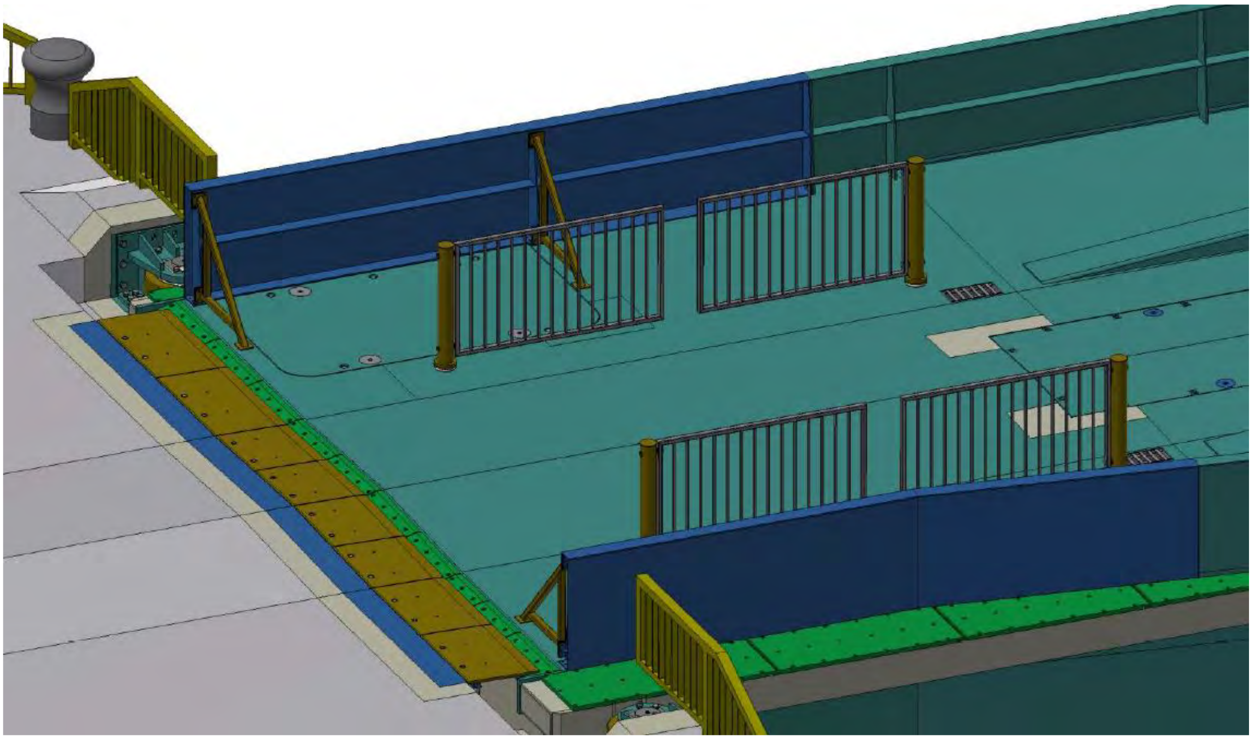

Figure 3-4 Flood protection line, gate skinplate increase

The design specifications were set by a wave study. Since DIN 19704 does not include any requirements for wave loads, EAU is used for this. The approach is conservative for large gates, since large lock gates deform more severely than embankments and therefore impact forces reach at lower heights. This results in the following loads for the sliding gate.

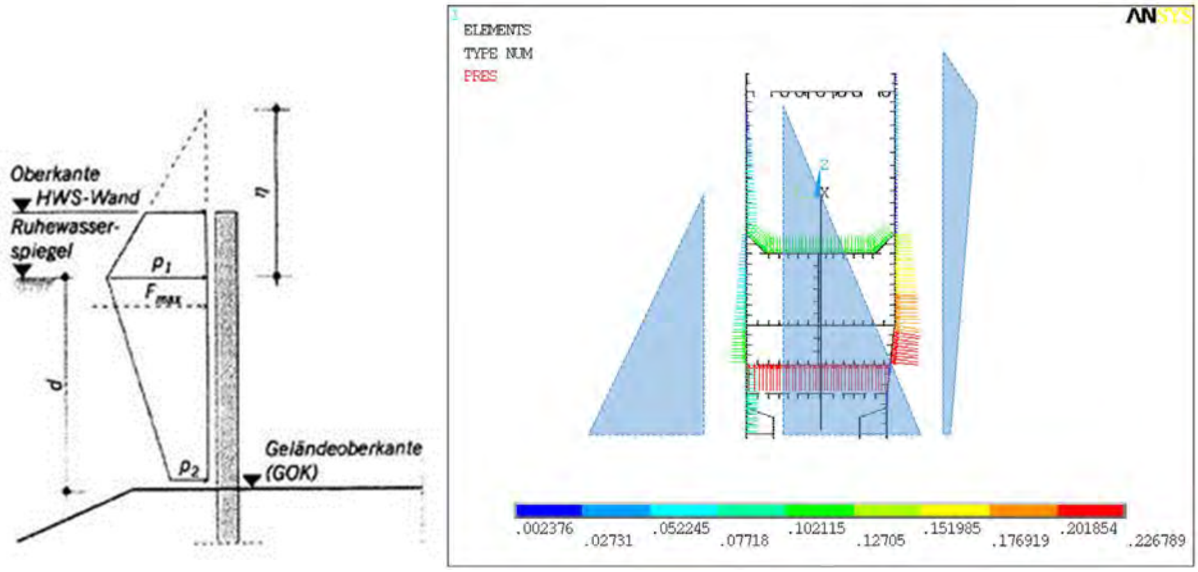

Figure 3-5 Breaker load (left) and implementation in computation (right)

4. DESIGN REQUIREMENTS AND REQUIREMENTS RESULTING FROM DIMENSIONING, INTERACTIONS AND EFFECTS

The manufacturing methods, safety requirements (e.g. design conception) and social requirements such as occupational safety have significantly changed since the construction of the large historic locks in 1914. With apparently the same key dimensions, this results in completely different detail solutions. Examples of requirements and their interactions are described below.

4.1. Safety requirements vs. floating depth

Due to the increasing values damaged in case of a failure, society requires increasingly high availability and hence safety for supporting structures, processes etc. (Schneider & Schlater, 2007). Higher safety requires the dimensioning of more robust structures; fatigue, for instance, was not investigated for the old gates at Brunsbüttel lock. For this reason, many structures are heavier than in the past.

In a sliding gate, this additional weight is proportionate to the general weight distribution, since all areas of a gate are affected to the same extent in a first approximation. The additional mass can only be compensated for by additional buoyancy cells to comply with the maximum possible floating depth and optimise the bearing loads on the supporting structure (upper carriage, undercarriage) and their distribution. If the floating depth of the gates is to remain the same, given the above-described “increase in mass”, the extra buoyancy can frequently only be achieved by buoyancy cells arranged towards the bottom of the gate. This adversely affects floating stability, which can only be compensated for by ballast positioned at the gate bottom. This additional ballast in turn requires extra buoyancy, i.e. the system is self-reinforcing reacting sensitively to general design conditions, please also refer to Section 4.3.3 Floating stability.

4.2. Occupational safety vs. buoyancy safety

One result of modified occupational safety requirements is that all accesses have to be wider and less inclined than in the past. As a consequence, accesses to machinery rooms need significantly more space. Since the staircase - as a hollow body - always increases buoyancy with rising water levels, the overall buoyancy of the sliding gate increases with growing staircase dimensions. In old gates, only ladders provide access to machinery rooms.

A material supply shaft with similar dimensions is required to supply completely assembled components into the machinery rooms inside the gate; by means of a mobile crane provided, this shaft can also be used to evacuate injured persons from the machinery room.

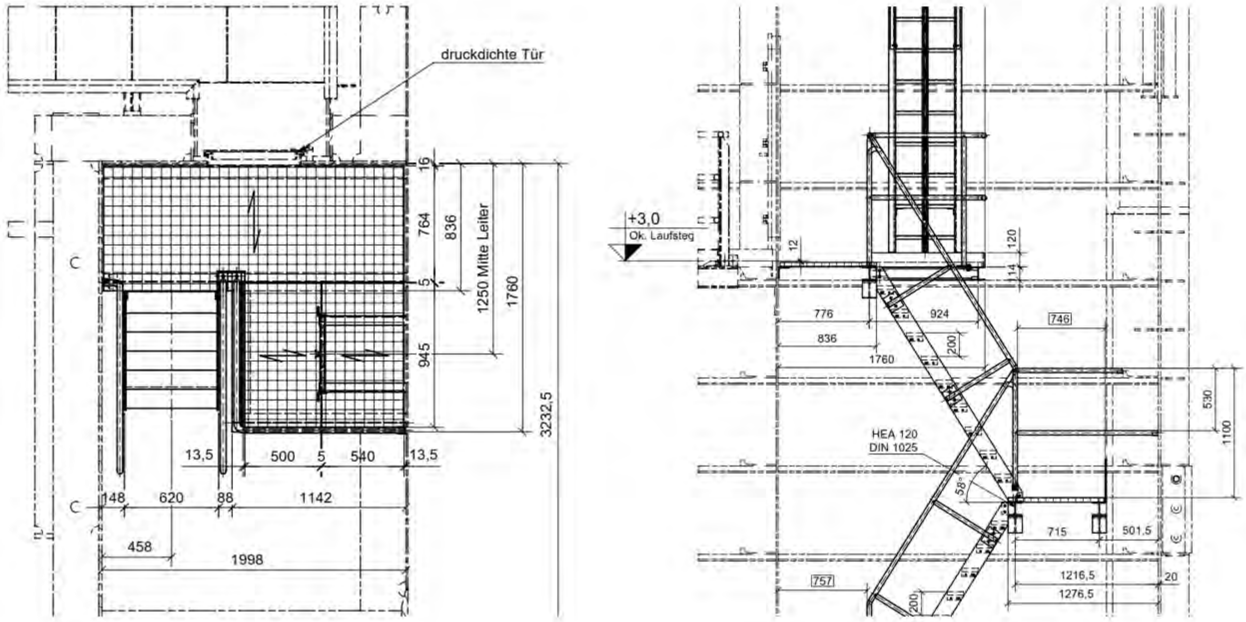

Figure 4-1 Staircase layout, cross-section

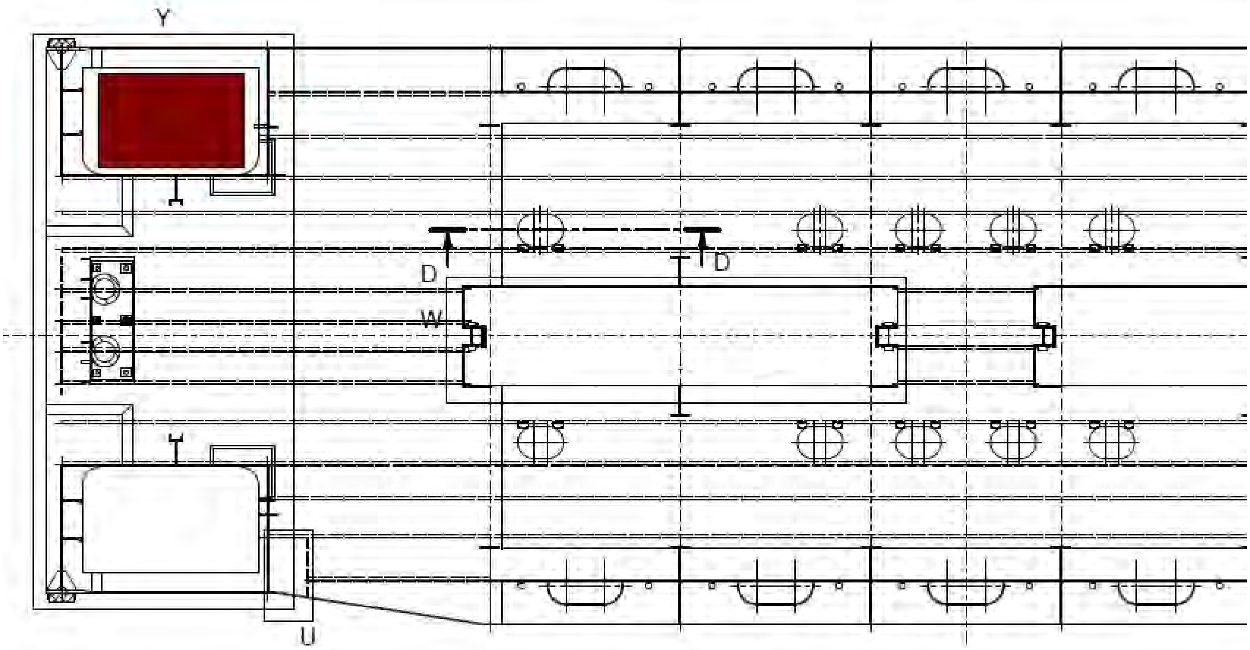

Figure 4-2 Material shaft with maximum transport sizes

The additional buoyancy described above has a negative effect with regard to the sliding gates' buoyancy safety during operation. It is to be ensured that the gate will not float at any outside water level. The additional buoyancy hence requires additional ballast to ensure the gate's anti-floating safety. This additional ballast is only partially compensated by the higher structural weights of the gate (cf. Section 4.1). The additionally required ballast can variably be provided via the ballasting/emptying system. To achieve an improved floating stability, the additional ballast can be provided as permanent ballast, e.g. Heavy concrete, at the very bottom of the gate, cf. Figure 4-3.

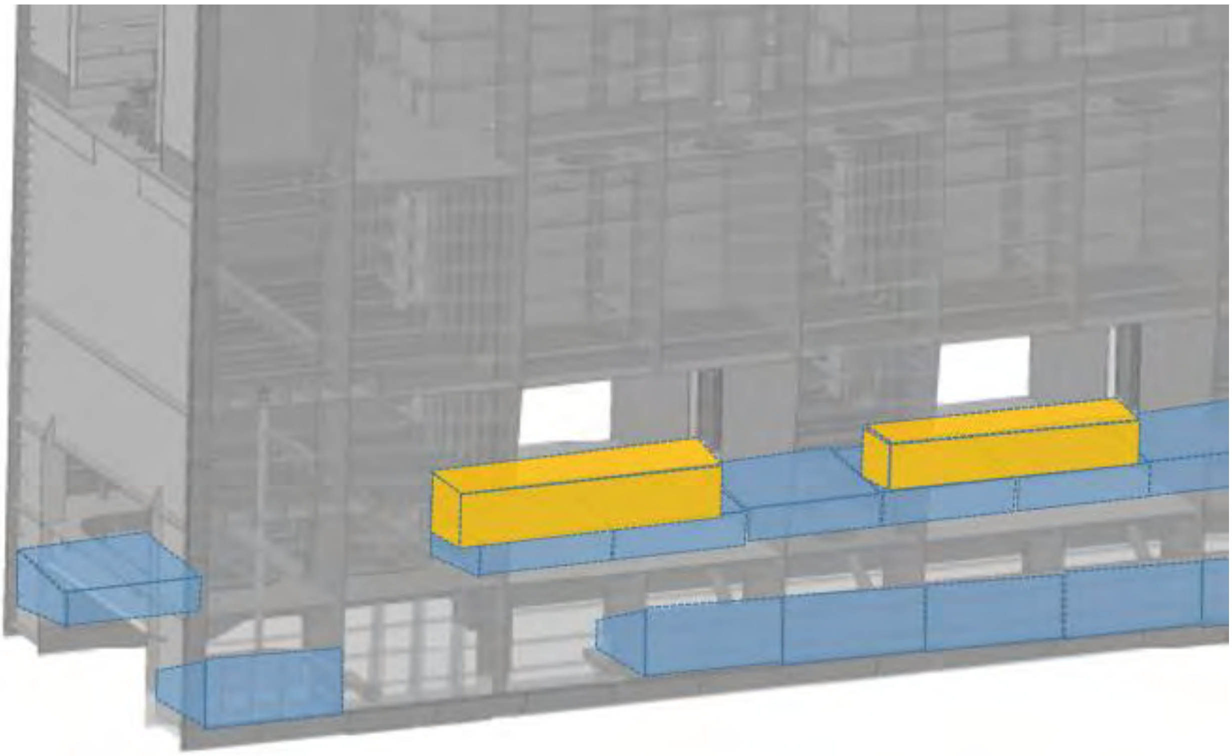

Figure 4-3 Identification of hollow spaces for permanent ballast (blue: methodically used/orange: reserve for planning)

The staircases have been arranged in the corners to achieve maximum stability for assembly and lowering the gates (analogous to disassembly and floating), see Figure 4-4. Due its the high Steiner's proportions when the tank ceiling immerses into the water, the gate will then maintain significantly more stability than a comparable gate with centrally arranged staircases.

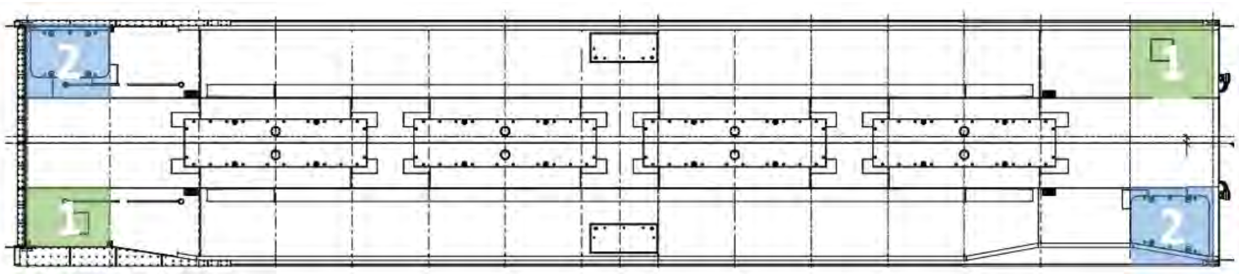

Figure 4-4 View on sliding gate (1 – staircase / 2 – staircase in corner)

4.3. Floating properties

The floating properties have an influence of some important parameters and act on the structure as well as the structure vice-versa acts on the floating properties. The floating properties can basically be differentiated by the following parameters.

4.3.1. Floating depth

In general, the floating depth has a bottom limit. The causes for this limit are e.g. the assembly/disassembly conditions (the gate must float above the lock sill and the already installed undercarriage), but also the fact that all locations near the lock must be reached at pre-defined water levels. The floating depth in consideration of the tide hence also determines the time slot available to lower the gate with a pre-defined minimum distance above the undercarriage). As soon as the gate has been swivelled above the undercarriage, the minimum distance is no longer required.

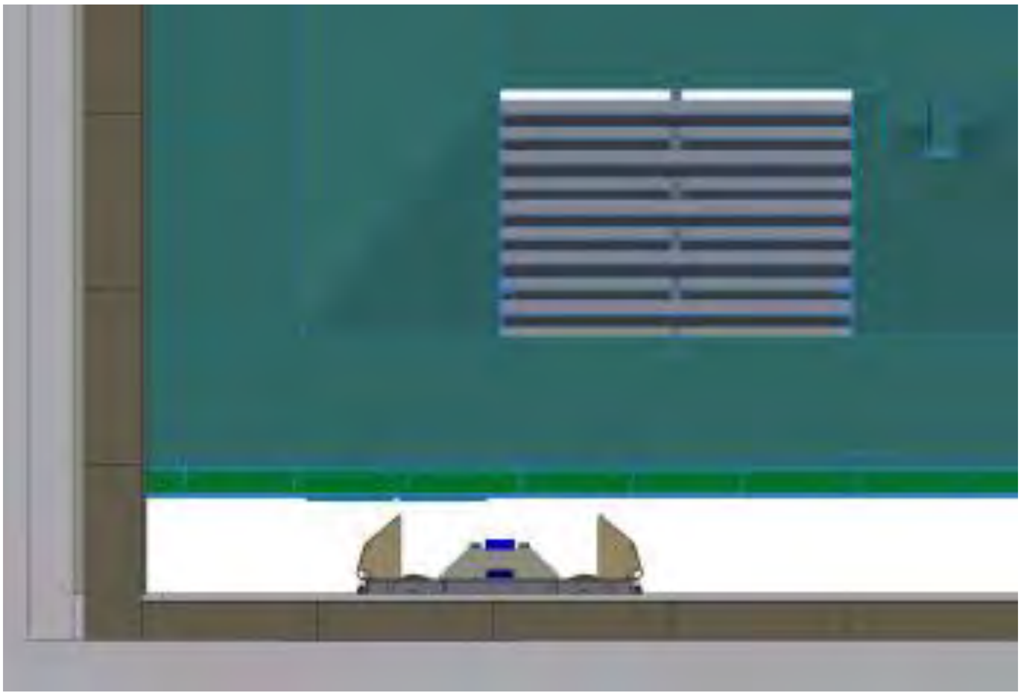

Figure 4-5 Presentation of gate floating above the undercarriage

The remaining time for installing the gate above the undercarriage by means of tugboats can be determined based on the tide speed. The process of swivelling in the gate should be started when the tide increases.

Two additional lifting pontoons are specifically scheduled for the 5SKB project. The lifting pontoons can be coupled to the floating sliding gates without the input of additional manpower. The lifting pontoons will generate additional buoyancy and hence reduce the gate floating depth. In addition, the floating stability of the gate-pontoon combination is significantly higher than of the floating gate alone. The lifting pontoons will not be addressed in more detail in this report.

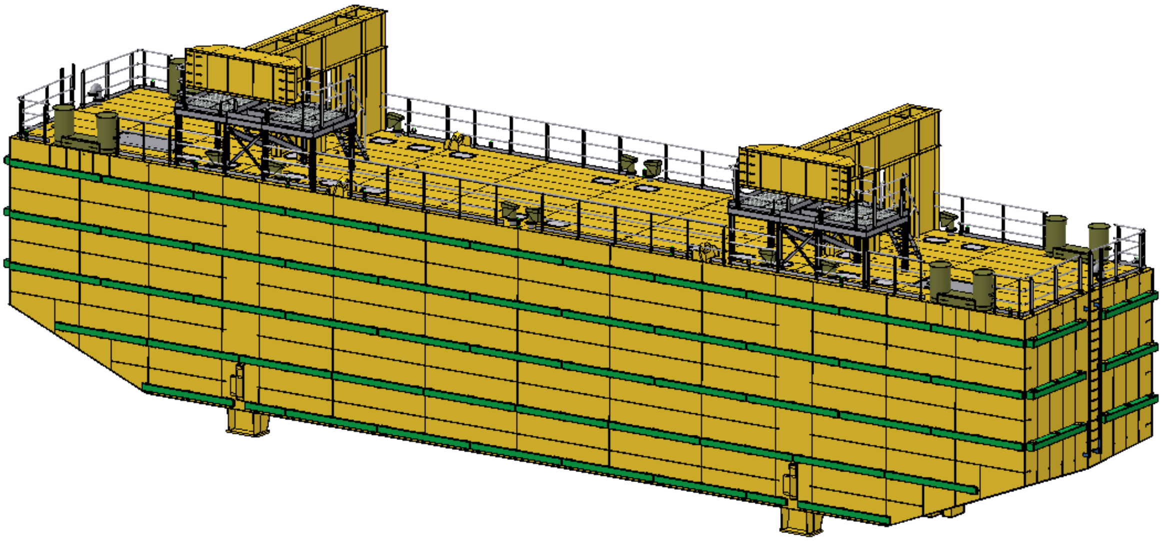

Figure 4-6 Presentation of lifting pontoon

4.3.2. Buoyancy safety vs. load discharge

Buoyancy safety is specified normatively in Germany, since a floating of the gates involves a significant storm tide risk for the cities beyond the flood protection line. This buoyancy safety does not depend on the structure size, the design methods and the manufacturing processes. In this case, a systematic statistical analysis to determine safety factory in consideration of all major parameters and their statistical distribution would be reasonable. In this respect, the selected depth of engineering as well as the manufacturing process should be taken account of.

The determination of the masses and the buoyancy volumes using a 3D-BIM model on as-built basis is much more accurate than the determination based on simple two-dimensional design drawings. Since safety factors always also take account of how actual components are represented by the numerical model, the required safety factors could for instance be reduced with an increasing accuracy of the methods.

The safety factors are the product resulting from the acting buoyancy (e.g. ballasting cells, machinery rooms, solid body buoyancy) and the counteracting driving forces [e.g. dead weight of fitted gates (in dependency on the method of weight determination), ballast (permanent and variable), variable loads such as silt load].

As regards varying ballasts (filling of ballasting tanks), process reliability must be included in the computation (automatic filling, monitoring of automatic filling, staff (and their qualification) for supervision vs. remote control only). In this respect, the probability for the stability of the gates as required by society should be verified; this verification should take into account that a loss in stability may result in considerable damage to the cities located beyond the dike protection line.

The key portions of the buoyancy forces are the buoyancy of all ballasting cells and of the machinery room. The size and location of the ballasting cells must be selected so that the entire gate floats safely on the one hand. On the other hand, the ballasting cell buoyancy must be countered with extra ballasting water including a safety margin in addition to the gate dead weight as well as all permanently installed equipment to meet the buoyancy safety of the installed gates.

The general determination of buoyancy safety in Germany results in comparatively high bearing loads for structures with a large mass. During operation, too, buoyancy safety must be ensured; for a specified safety of over 1.10 (=R_d / E_d), for instance, a minimum of 10% of the overall mass of the gate must be discharged through its support structure (upper carriage / undercarriage and their load transfer e.g. into the wheel-rail system). As a consequence, the relevant stress of load discharge in the vertical direction result from buoyancy safety requirements.

4.3.3. Floating stability

Floating stability criteria have to be observed for the floating gate so that it meets occupational safety requirements despite waves and wind. The stability criteria comprise the numerical inaccuracy of the difference of high numbers (buoyancy and downforce are equal with a floating gate; balance of forces). Manufacture therefore requires great care and is best with the weighed masses of the fitted gates or a mass computation during workshop design equivalent to weighing; otherwise the target values (e.g. floating stability, floating depth, etc.) cannot be achieved.

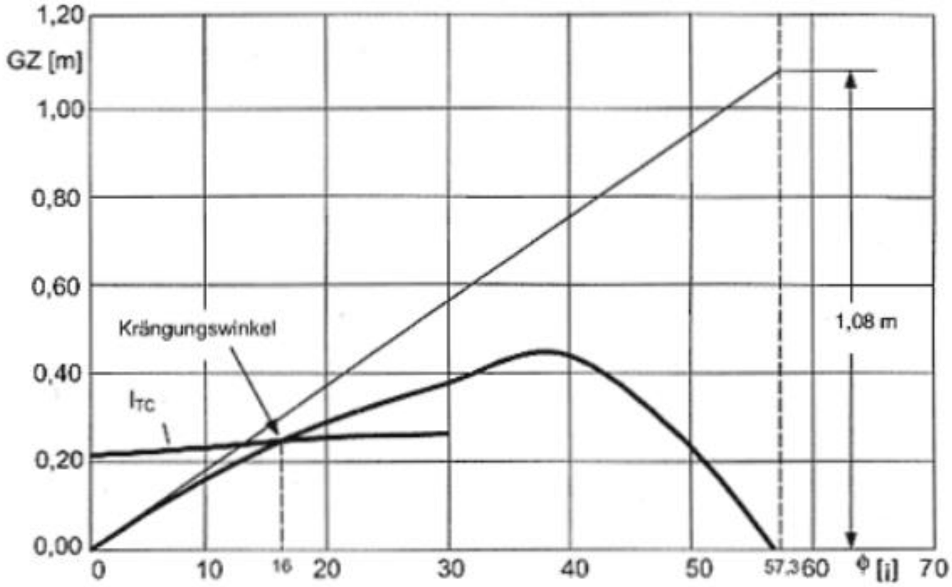

Figure 4-7 Example for lever arm curve with presentation of metacentric height (1.08 m at 57.3°)

In general, floating stability is determined by metracentric heights. In addition, floating stability properties are described by requirements of lever arm curves. The connection for rectangular cross-sections may be described as the reflection of the source tangent on the floating lever arms when the the metacentric height is plotted at 57.3° (= 1.0 rad).

$$h_M = \frac{I_0}{V_W} - h_k \qquad (3)$$

In this equation, I_0 is the moment of inertia of the submerged area around the axis for which the floating stability investigation is to be made. The submerged volume results from the weight of the submerged body. The result is:

$$V_W = \frac{G_{Tor}}{\gamma_W} = V_{Schwimmtank} + V_{Rest} = A_{Tank} \cdot t_{Tank} + V_{Rest} \qquad (4)$$

G_{Tor} - the total dead weight of the gate with all equipment. All other weights such as contamination, vegetation, are to be analysed in the relevant combinations

γ_W the gross density of the water

$V_{Schwimmtank}$ the volume of the floating tank submerged into the water

V_{Rest} the residual volume of all other bodies submerged in the water; with a sliding gate, this is primarily the volume of all steel cross-sections below the floating tank

A_{Tank} the surface area of all buoyancy cells

t_{Tank} the tank submersion depth

The submersion depth is of great significance, since it determines a major parameter of the buoyancy centre of gravity position. This is required to determine h_k. h_k is the elevation difference between the centre of gravity of the dead weight and the buoyancy an can be computed as follows:

$$h_k = a_G - a_v$$

$$h_k = \frac{\sum G_i \cdot e_i}{\sum G_i} - \frac{\sum A_i \cdot e_{A,i}}{\sum A_i} = \frac{\sum G_i \cdot e_i}{\sum G_i} - \frac{\gamma_W \cdot \sum A_i \cdot e_{A,i}}{\sum G_i} \tag{5}$$

$$h_k = \frac{1}{G_{Tor}} \cdot \left(\sum G_i \cdot e_i - \gamma_W \cdot \sum A_i \cdot e_{A,i}\right)$$

All elevation data refer to the bottom point, i.e. the centre of the floating tank in this case. The individual centres of gravity are multiplied by their lever arms $\Sigma G_i \cdot e_i$ and $\Sigma A_i\, e_i$. These figures, which are subtracted from each other, are considerably higher than their difference. This means that if one of the two figures only fluctuates a little, which they always do due to tolerances, the effects on the result will be enormous. This requires great care in material ordering, in case of changes, during assembly, etc., so that the floating stability reaches and acceptable value.

In contrast to large vessels, the description of the outside shape of the sliding gate bodies displacing water is much more difficult. The reasons for this are that the shape is more rugged and frequently not symmetrical along the longitudinal axis. The asymmetry of the 5SKB results from the floating process requirements. The skinplate is located on one side, only, to provide for enough space for floating.

The load from variable loads on gates does not result from the load masters (vessels), but partially by accident from natural processes such as silt settlement, etc. or from the system operation. Staff is not continuously present on site, but the lock is controlled from a control station. An effect from unequal inclinations (trim/heeling) may be detected by installed measuring systems on the 5SKB, for instance.

The sensitivity is presented based on the intermediate results of the floating stability computation for the 5SKB sliding gate. The graph can be taken as an extreme example; if the buoyancy centre of gravity a_V shifts downward by 10%, the metacentric height and hence the floating stability is reduced by 50%. The sensitivity of floating stability shows that workshop design and manufacture require extreme care, since for instance the provision of additional or reduced masses cause the gate to float higher or lower. For this reason, manufacture requires great care. With sufficient counteraction in workshop design, a weight computation as weighed must be performed.

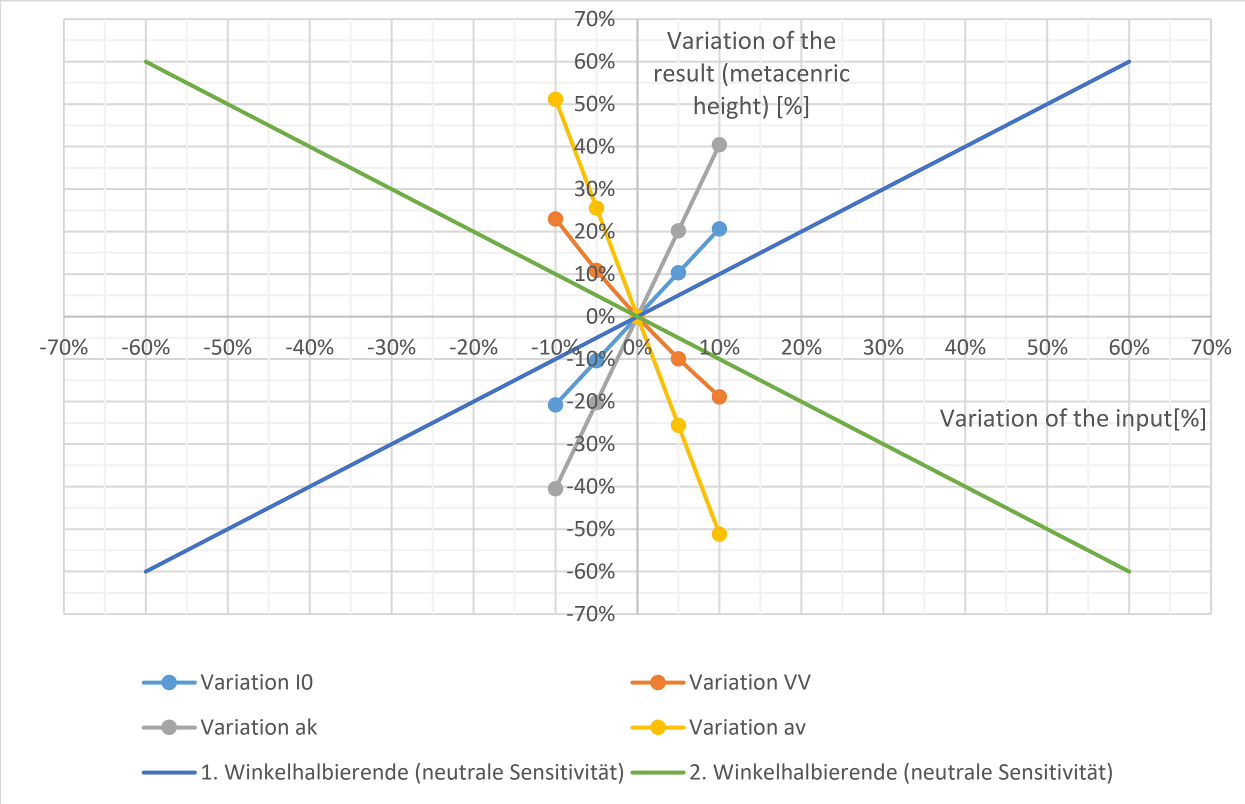

Figure 4-8 Sensitivity analysis of input values and their effects on the metracentric height

All parameters mentioned are actually subject to tolerance; this applies to both the design and the manufacture of the steel structure (e.g. rolling tolerances, deviations in individual dimensions, auxiliary assembly equipment), and the coating thickness. Apart from this, all variable effects such as silt, contamination or vegetation are difficult to assess and are inaccurate. The buoyancy centre of gravity, too, is subject to uncertainties despite very careful design, e.g. by accidentally enclosed air.

Based on the metacentric height, the raising moments can easily be computed for slight changes in the floating position:

$$M_A = G_{Tor} \cdot h_M \cdot \delta\varphi \qquad (6)$$

This equation applies to the surroundings of the origin of the tared floating body. The resisting moment from the floating body (pantokarene) is opposed by the effects from wind, waves, skew during tugging, etc. These effects shall be considered for lock gates for the exact design conditions, e.g. the transport to the berth and the travel distance to this location. In comparison to vessels on the open sea, these swell parameters are appropriately lower.

4.4. Filling valves

The filling vales shall be operated by hydraulic cylinders. For each valve channel, two rows of valves are provided. The valves are of a double-seal design so that the valve channel can also be sealed by one row of valves, each. In addition, the valves are self-closing types to provide for sufficient process reliability for closing Kiel Canal. For this reason, the filling valves were ballasted so that they also close in the absence of electricity. As a consequence, sufficient process reliability can be ensured to be able to protect the interior against storm tides in case of a power failure or damage to the hydraulic cylinder.

Regarding floating stability, one of the key parameters is the elevation of the valves, since they have an outstanding effect on floating stability due to their comparatively high dead weight. These and other parameters are considered in the determination of the reliability of the gate system.

4.4.1. Filling valve design requirements

Flow velocity is significant for smaller vessels (whose floating position should not excessively be affected), tie-up forces and scour protection, as well as for the static, dynamic and fatigue rating of the valves. A key factor for safe filling is therefore the structural and flow-related design of the filling channels. Angle sections (so-called disturbing angles) and are used in the filling channel to convert energy and to break up the flow into the lock chamber. In this way, the water flow is split up, homogenised and hence the maximum flow velocity is reduced.

Apart from the water level difference itself, the flow direction depends on different specific water weights in the lock chamber and on the opposite side (inside head: Kiel Canal; outside head: Elbe). Consequently, it was examined whether fresh water flows into brackish water or brackish water flows into fresh water (see Figure 4-9). The illustrations in the following figures show the computation results with the assumption of unrealistic specific water weights, pure fresh water is not found at the lock, no more than the salinity used here. However, the effect becomes evident based on this design specification. The realistic salinity has a considerable influence on the tie-up forces (Thorenz, Septmeber 18-21, 2017); in practice is has shown that no problems are to be expected for sufficient redundancy is provided for.

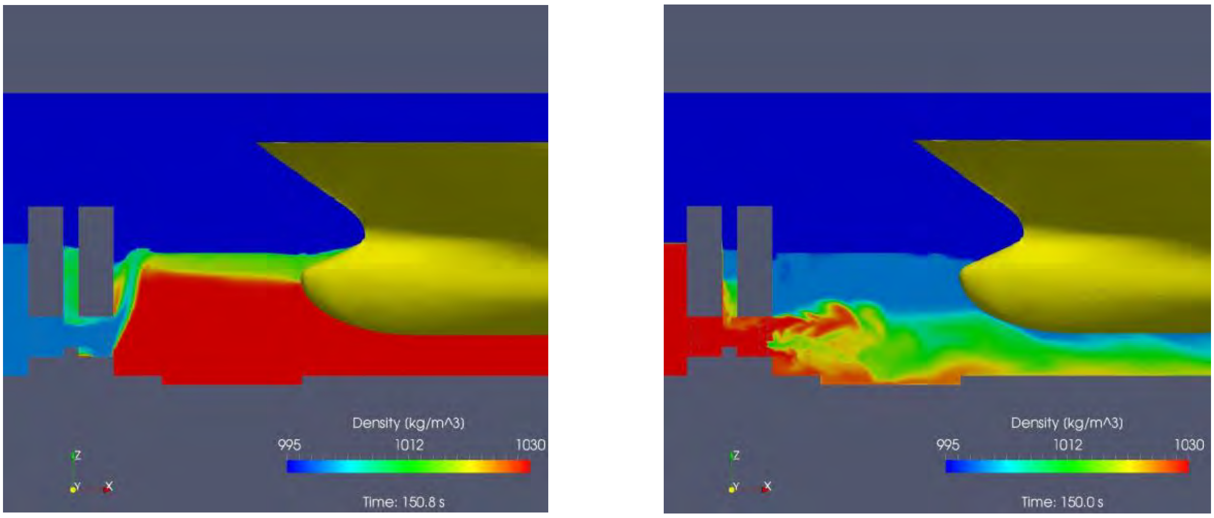

Figure 4-9 CFD simulation, effects of different specific water weights

Due to the sensitivity of Teflon coating against abrasion on the seals, the optimisation of the gates' lifting forces was omitted to that effect. The seal types used were conventional music note profile seals. With respect to sealing, the deformation of the gates, the supporting structure and the seals have to be examined. Where flow has a significant influence on the gap topology and therefore the flow behaviour, coupled mechanical-hydraulic (cfd-computational fluid dynamics) computations are required.

Due to the immense computation effort, it is neither reasonable nor necessary to make these computations for the entire system (complete gate), but subsystems are used. These subsystems require the identification of appropriate interfaces. The correct transfer of general conditions is needed at these interfaces. All tolerances and distances must be defined in a way that the sealing as required by the Client is ensured, but jamming of the seals or excessive wear is avoided. There is a certain probability for vibration and abrasion-induced damaged. The structural and flow-related design of the filling valves are essential factors for the overall lock's function.

4.4.2. Filling valves - facility management planning

The maintenance and repair of the filling valves is a significant aspect of design. Areas where mobile cranes may discharge their forces are provided on the accessible top of the gates. For this reason, facility management has already been planned along with the actual design. As a graphic example, the installation sequence of the filling valves, which may be dismantled from the gate for maintenance purposes, is shown. Figure 4-10 shows an overview of the filling valves and there mounting.

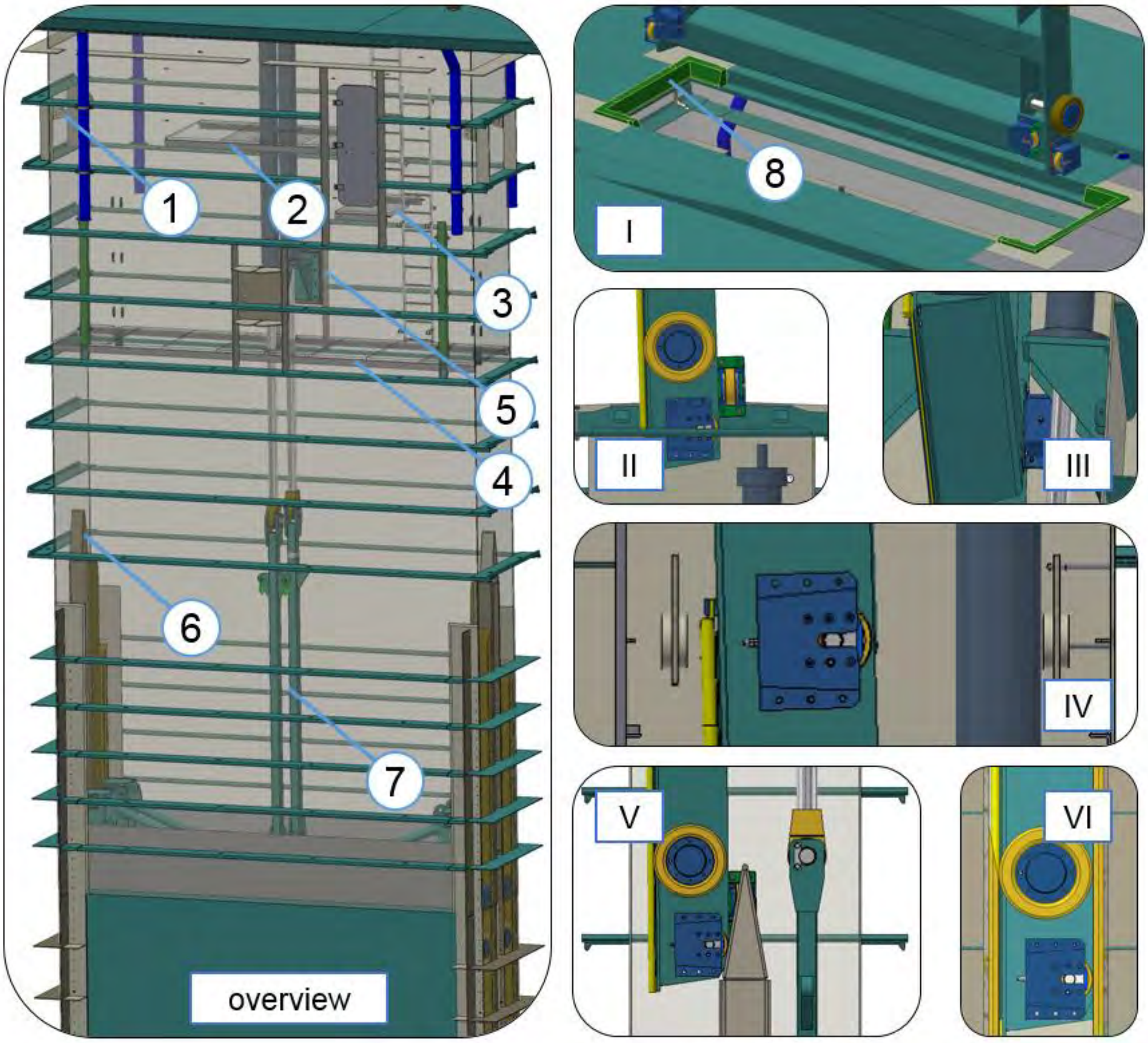

Figure 4-10 presentation of filling valves and important points during installation

Figure 4-10: **overview)** shows the final stage, Position is defined as distance between roadway and lower edge of filling valve (FV), **I)** FV above roadway, **II)** FV 0.5 m beneath the roadway - visual check during positioning - attention: hydraulic piping, **III)** FV 4.90 m beneath the roadway – bottleneck cylinder console to be observed, move slowly, **IV)** FV 5.66 m beneath the roadway - bottleneck at seal, shackle (visual check), **V)** Position 9.36 m beneath roadway Guide installation, **VI)** Position 11.84 m beneath the roadway - Visual check, spring-loaded roller engages, **1)** lifting eye, **2)** platform cylinder hydraulic, **3)** platform entrance, **4)** platform cylinder console, **5)** cylinder console, **6)** guide installation, **7)** coupling bar, **8)** temporary guiding installation (UHMW-PE)]

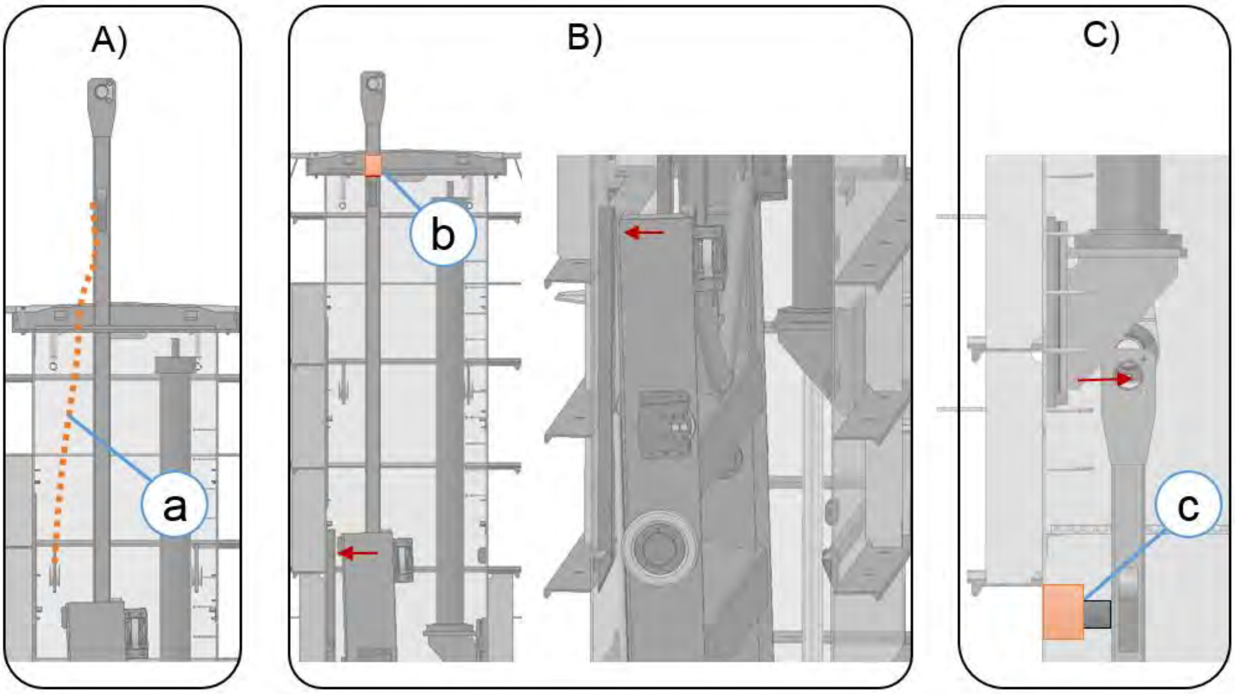

Figure 4-11 presentation of filling valves and auxiliary construction for installation

Figure *4-11*: **A)** FV 7.25 m beneath the roadway - attachment of securing rope, **B)** FV 8.40 m beneath the roadway - observe position mark – deviate FV by crane to contact with UHMWPE keys for assembly , **C)** FV 14.00 m beneath the roadway - held by securing ropes (a) at accurate position – deviate coupling bar by hydraulic cylinder (connection cylinder with coupling bar), **a)** securing rope, **b)** position mark, **c)** hydraulic cylinder

4.5. Flushing system

In combination with the ballasting/emptying system, a flushing system was provided for the 5SKB. The flushing units are located in the tank ceiling and near the undercarriage rails. The flushing system's task is to clear both the area of the undercarriage rail system and especially the tank ceiling of the gate from sedimented silt. . In addition, a pump system floods and/or empties the ballast tanks. This requires extensive piping for this pump system on and in the gate. The control units for these components are partially arranged on the gate. The provision of a machinery room in the gate for accommodating all these units is necessary.

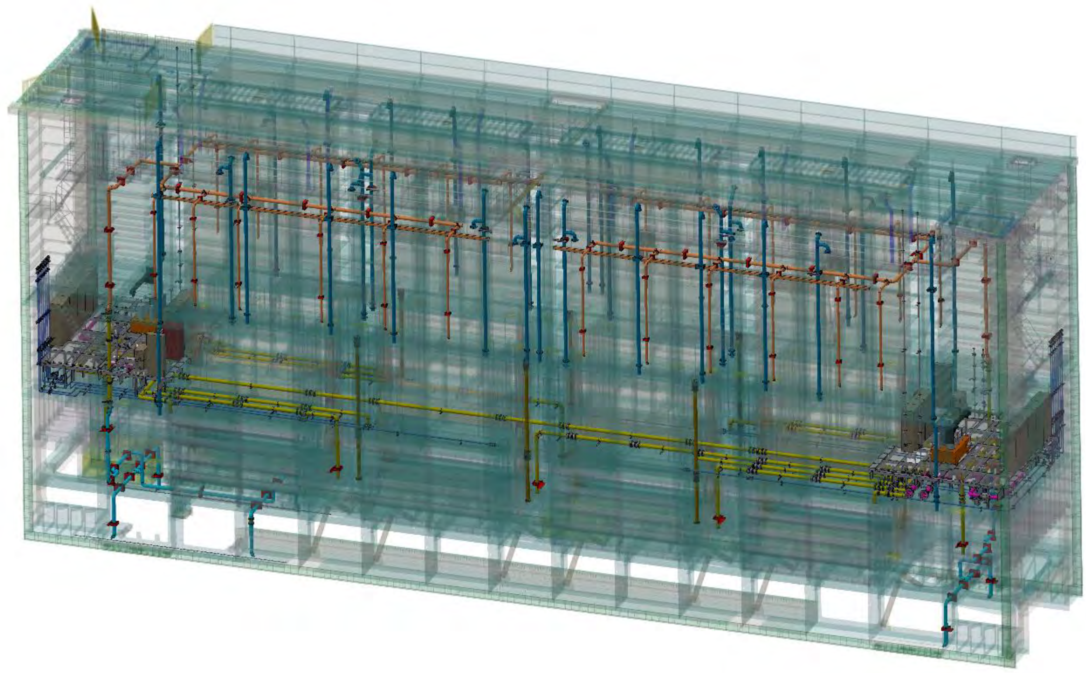

Figure 4-12 Gate flushing system (orange pipes - tank ceiling / blue pipes on gate bottom edge - rail system)

5. ABSTRACT

Due to the variety of requirements to be met, the design of sliding gate structures for a sea lock is very difficult (one of the most difficult design tasks). A number of individual disciplines is addresses, i.e. steel structure, shipbuilding, piping construction, mechanical engineering, control engineering, automation. The design is best performed in a three-dimensional BIM taking account of all disciplines. The numerical representation must be sufficiently transparent so that the different conditions of such a gate may be imaged.

The successful design and manufacture of such complex works requires the cooperation of all parties involved.

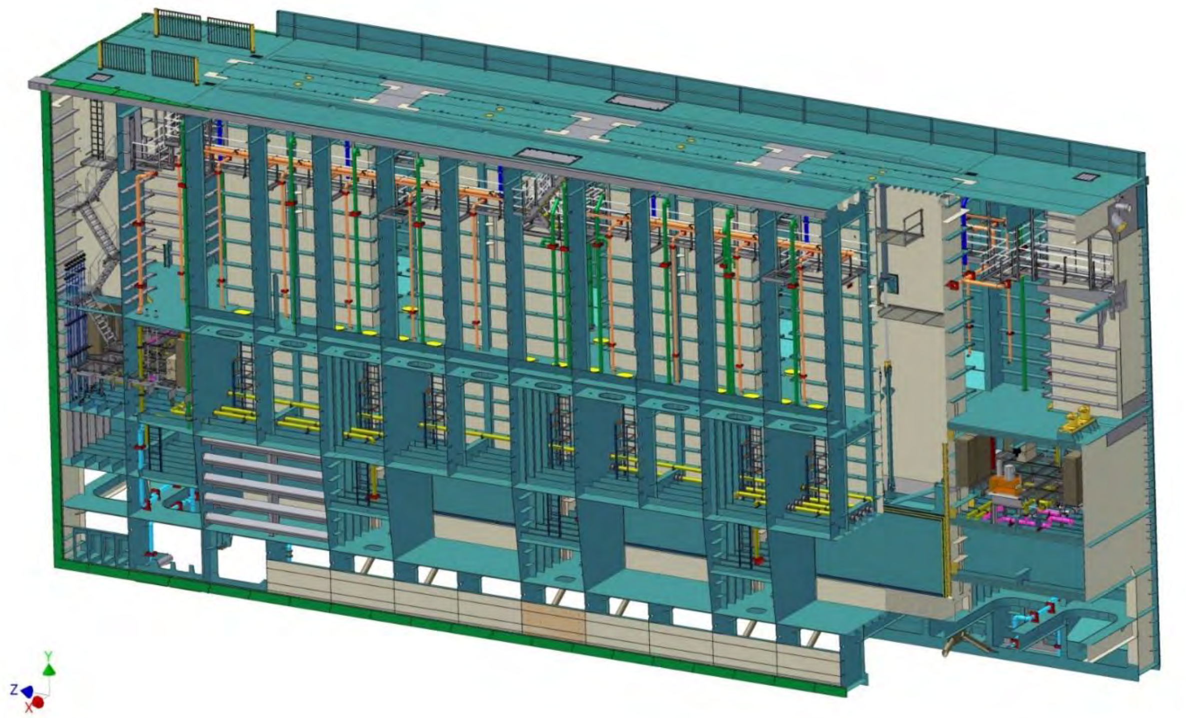

Figure 5-1 5SKB isometry CAD model

6. LITERATURE

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FLOW-INDUCED VIBRATIONS AT HYDRAULIC STRUCTURES

by

Michael Gebhardt*[1]*, Georg Göbel, Martin Deutscher, Walter Metz and Carsten Thorenz

ABSTRACT

Despite of remarkable research during the last decades, such as Naudascher and Rockwell (1994) or Kolkman and Jongeling (2007), flow-induced vibrations still cause problems in hydraulic engineering. Vibrations occur not only at old gates, but also at new gates, such as radial gates or filling valves at miter gates. Against this background a research and development project was initiated in the Federal Waterways Engineering and Research Institute in order to analyze the different causes by field measurements and by the use of numerical models. The aim is to identify the excitation mechanisms and to improve the current construction standards (Göbel et al., 2018). Studies are presented, where gate or gate parts were excited to vibrations by fluid-structure interaction. Typical frequencies are presented for the vibration of hydraulic gates, for spring supported sealing systems and for rubber seals. The frequencies are helpful for operators to identify the source of vibration in order to initiate constructive or operational improvements.

Due to the relatively high elasticity of long span gates, bending vibration might occur, if the gate is excited to vibrate by under- or overflow (Ishii and Knisely, 1992). One example is a weir on the River Weser in Northern Germany, where vibrations could be observed after opening the gate for a few centimeters. A resonance frequency of 1.5 Hz could be determined for the 42 m wide lifting gate. In an experimental testing a predominant bending vibration mode could be identified with a characteristic wave pattern upstream of the gate (Fig. 1a). Numerical studies were used in order to identify the critical opening widths in combination with the downstream water level.

(a) (b)

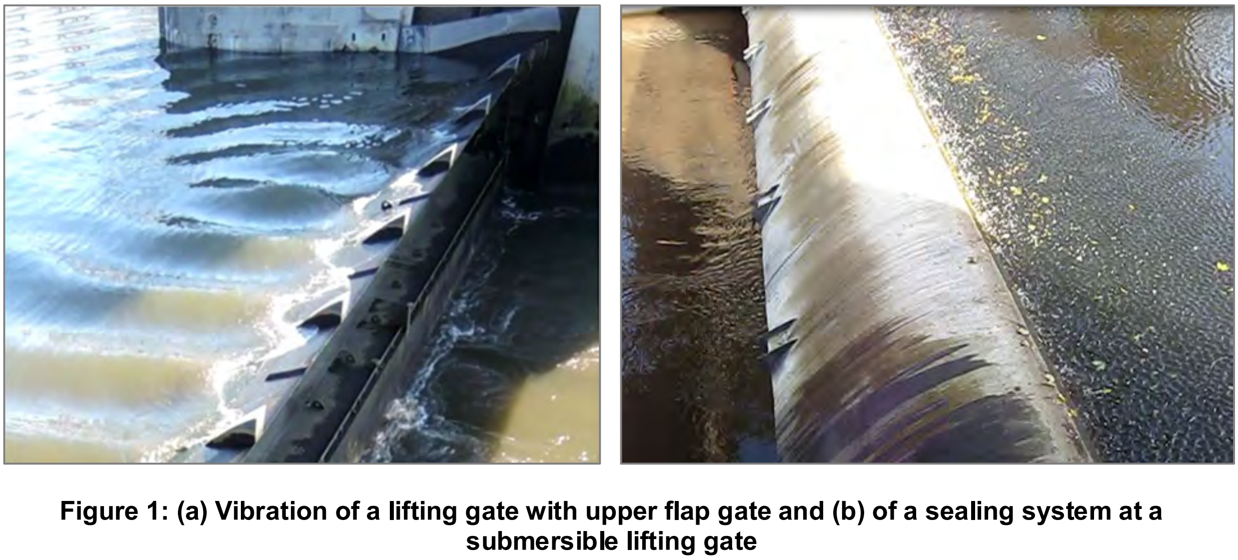

Figure 1: (a) Vibration of a lifting gate with upper flap gate and (b) of a sealing system at a submersible lifting gate

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Typically, gates vibrate in a single-digit frequency range (Fig. 2b & d). If underflow gates are equipped with spring supported sealing systems, where the sealing acts against the water pressure, they generally tend to vibrate in closed or slightly opened position because they are elastically mounted. These constructions are pretty diverse and thus the resonance frequency may vary between the different systems. Experiences show that the resonance frequency is significantly higher in a range between 15 and 40 Hz. Figure 1b shows the wave pattern caused by a vibrating spring-supported seal system.

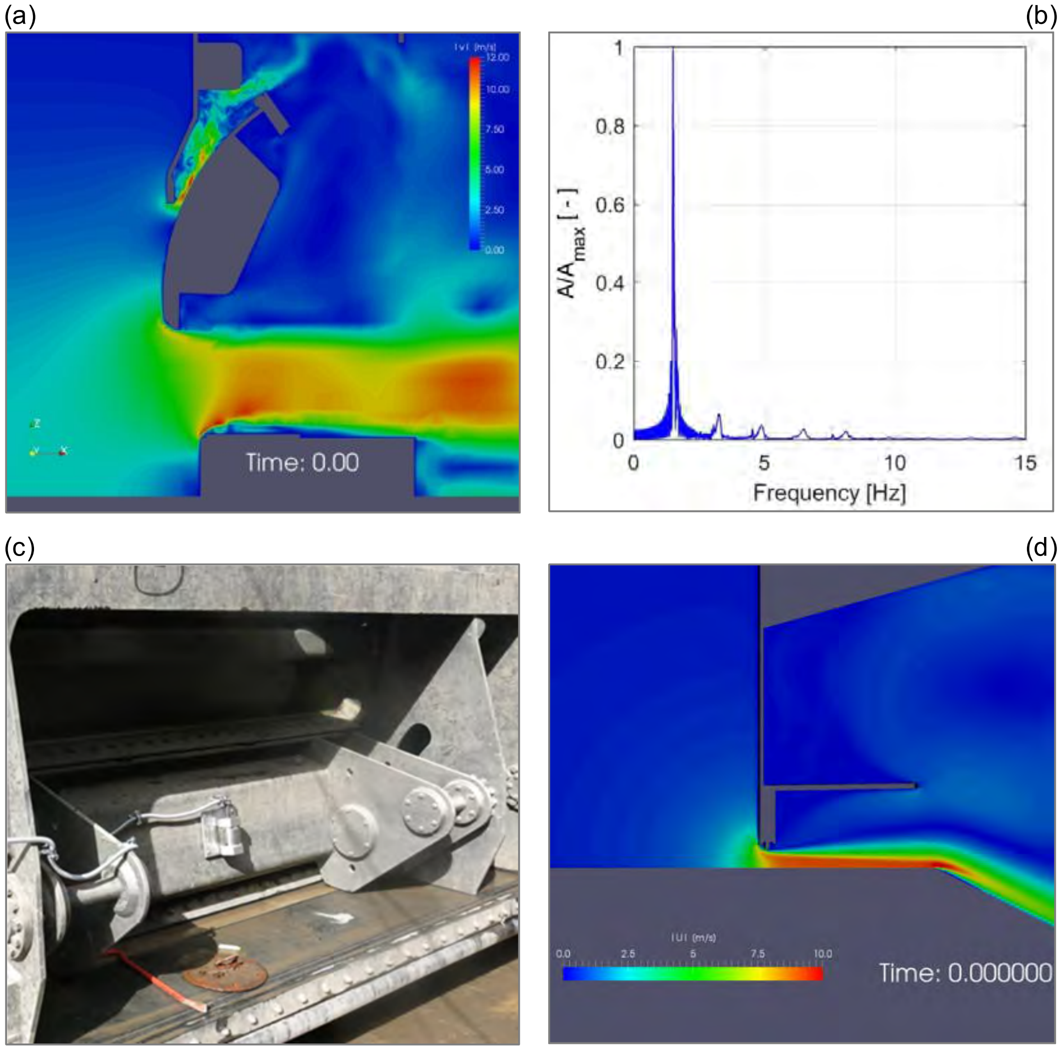

Figure 2: Applied methods: (a) numerical simulation of a filling valve; (b) Frequency analysis of a measuring signal; (c) acceleration sensor at filling valve of a mitre gate and (d) numerical simulation of fluid-structure interaction of a lifting gate

Seals in the shape of a musical note have multiple advantages if they are installed properly. The flexibility of the rubber makes the seal adaptive to uneven surfaces and tightness can be improved. Naudascher and Rockwell (1994) mentioned already that J-seals can also be a source of flow-induced vibrations during small openings or leakage. Seal vibrations occurred at a new miter gate on the river Neckar (Fig. 2a). During commissioning heavy vibrations occurred when the filling valve was opened a few centimeters. Intensive in-situ measurements were carried out to identify the source of vibration (Fig. 2b). Sensors were applied on different positions of the radial gate which showed a matching dominant frequency of 40 Hz. By comparing the signals it could be concluded that vibration was induced at the top sealing. Therefore, the construction was improved. Another example of seal vibrations is a new radial gate with upper flap gate. During commissioning loud humming vibrations were noticed at small opening widths. In order to determine the resonance frequency of the vibrations, the sound track of a video was analyzed. The dominant frequency was 37 Hz, which fits quite well in the known frequency range of 35 to 50 Hz for vibrating J-seals. After removal of the J-seal, the buzzing vibration was gone.

In addition to field measurements and physical models numerical simulations of fluid-structure interaction (FSI) is a very promising method to analyze problems and to improve constructions. First applications are presented. Typically, two standalone solvers for the fluid and for the solid region are used. On the interface of the two regions, an information exchange between the solvers is required. Viscous and pressure forces from the fluid region will be set as boundary conditions for the solid region and deformation of the solid region will be set as boundary condition for the fluid region. This method is called partitioned approach and is used in the solver package fsiFoam that can be included in OpenFOAM® (Göbel et al., 2017). Numerical methods are advantageous because they allow the evaluation of flow parameters at locations which are hardly accessible on site or even in a physical model.

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SIMPLE BUT ACCURATE CALCULATION METHOD FOR VESSEL SPEED IN A MINIMUM CAPACITY LOCK

by

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ABSTRACT

The increase of vessel size puts pressure on the maximum allowed vessel dimensions in various inland waterways and locks. Waterway authorities are increasingly challenged to allow vessels with dimensions that exceed the design specifications of the lock. Besides the risk of collision with the sill or the lock head, there is also the aspect of the time required to sail in and out of the lock.

This paper presents a desk study, prototype measurements, and verification and calibration of a simple but accurate calculation method based on the Schijf`s method for vessel speed (and squat) in the very confined water (blockage ratio: $k< 0.7$) in a minimum capacity lock. The calculation method has been verified with prototype vessel speed measurements from five locks and one ship lift.

As such this paper is a contribution to the understanding of the consequences of larger vessel sizes in the existing lock infrastructure, focussing on the vessel speed during sailing in and out of the lock and the subsequent increase of the lock-cycle time.

1 Introduction

1.1 Enlarging of the Twentekanaal and Lock Delden

The Twentekanaal in the eastern part of The Netherlands was opened in 1933. Originally the canal was constructed for CEMT Class IV vessels with a draft of 2,6 m. After more than half a century the Dutch government decided to upgrade the canal to CEMT Class Va vessels. In 2010 the 1st Phase of the enlarging of the Twentekanaal was completed. The 1st phase covered the western part of the Twentekanaal, some 30 km from the River IJssel up to Lock Delden, see Figure 1.

The 2nd Phase of the enlarging of the canal (from Lock Delden up to Enschede and the branch to Almelo) has been scheduled for the next decennium.

This enlarging comprises the widening and deepening of the canal profile, but does not include Lock Delden.

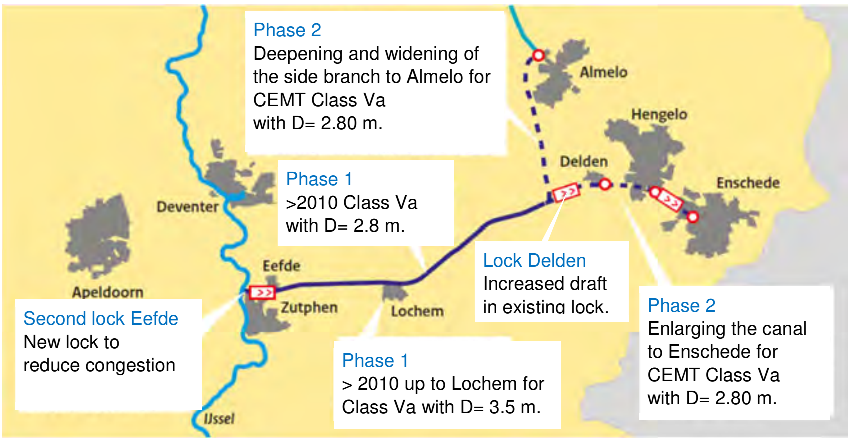

Figure 1: Situation Twentekanaal in the eastern part of the Netherlands

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1.2 Investigation Twentekanaal after completion of Phase 1

After the opening of the 1st Phase the pressure of the shipping industry grew to allow Class Va container vessel in the enlarged canal. In view of this demand, the authorities investigated the possibility of Class Va vessels with draft limitation. Since the cross-section of the canal east of Lock Delden has not yet been increased, the specialist of Rijkswaterstaat advised not to increases the maximum wet cross-section of the vessels above the existing 25 m² (=9.5 m * 2.6 m, corresponding to the beam and max allowed draft of CEMT Class IV vessels) as this would cause significant wearing of the slopes of the canal.

Maintaining the maximum wet cross-section of 25 m², results for Class Va vessels in a maximum allowed draft of 2.2 m (=25 m² / 11.4 m). The authorities followed this advice and in 2008 Class Va vessels were allowed with a maximum draft of 2.2 m. For Lock Delden this increase of the vessel class resulted in a significant decrease of the minimum beam clearance, whereas, the maximum blockage ratio in the lock remained unchanged. The minimum beam clearance forces the skipper to line up accurately for the lock and to sail with great care. However, since the blockage ratio of the vessel in the lock remains unchanged, the new access policy had hardly any impact on the lock entry and departure speed, and therefore, on the lock-cycle time.

1.3 Investigation Lock Delden for Phase 2

After the completion of the 2nd Phase of the enlarging of the Twentekanaal the maximum draft for the Class Va vessels will be increased to 2.8 m. In 2016 an investigation was started on the effects of the increased draft on Lock Delden. The first findings of the desk study are summarized in Table 1.

Lock and vessel			1933		2008	Future	Minimum lock	
Dimension and clearance			**Lock**	**Class IV**	**Class Va**	**Class Va**	**RVW(2017)**	
Length of lock	L_{lock}	[m]	133				125	
Length of vessel	L	[m]		85	110	110		110
Length clearance	LC	[m]		48	23	23		15
Lenght clearance ratio	k_{LC}	[%]		56%	21%	21%		14%
Width of lock	W_{lock}	[m]	12				12.5	
Beam of vessel	B	[m]		9.50	11.40	11.40		11.40
Beam Clearance	BC	[m]		2.50	0.60	0.60		1.10
Beam clearance ratio	k_{BC}	[%]		26%	5%	5%		10%
Depth of lock	D_{lock}	[m]	3.34				3.5	
Draft of vessel	T	[m]		2.60	2.20	2.80		2.80
Underkeel Clearance	UKC	[m]		0.74	1.14	0.54		0.70
Under keel clearance ratio	k_{UKC}	[%]		28%	52%	19%		25%
Blockage ratio	k	[%]		62%	63%	80%		73%
Schijf's method: limiting speed	v_{lim}	[m/s]		0.77	0.74	0.29		0.46
operational speed	v_{op}	[m/s]		0.65	0.63	0.25		0.39
water level depression	z_{op}	[m]		0.20	0.19	0.11		0.15
Lock entry time	T_{entry}	[s]		204	212	542		322
	T_{entry}	[min]		3.4	3.5	9.0		5.4

Table 1: Lock dimension and increasing vessel dimensions and lock entry time

With the acceptance of Class Va with draft of 2.8 m two of the lock clearances and the blockage ratio (indicated in Table 1 in red) are beyond the values recommended in the Waterway Guidelines (RVW,2017) for a minimum capacity lock.

PIANC (2015) states, that if the blockage ratio is 0.75 or higher, special measures have to be taken for safe passage. A vessel that enters a lock with a large blockage ratio and a high speed creates a wave in front of her bow. This wave propagates through the lock and is subsequently reflected at the lock gate. The returning waves can become significantly high, that the entering vessel is pushed back in reverse direction.

In addition, PIANC (2015) states that with small clearances, the entry and exit times are generally longer and depend increasingly on pilot skills, assistance by bow thrusters and the availability of guiding structures.

Questions that arise for situation with Class Va vessels in Lock Delden are:

- Is the UKC still positive and safe? Or does the vessel touch (or even damage) the lock sill?
- Is the vessel able to sail in and out of the lock with acceptable speed?
- Is the increase of the locking cycle time acceptable?

To investigate these questions a desk study on the lock entry speed and lock-cycle time has been performed (RWS-ON,2016) and the impact for Lock Delden has been reported (RWS-PPO,2016).

1.4 Previous investigations on vessel speeds in locks

In an investigation on the hydraulic forces on mitre gates, Vrijburcht (1994) compared the approach speed of vessels entering a lock, with the limiting speed according to Schijf's method (Schijf,1949 and Jansen and Schijf, 1953). He analysed measured vessel speeds and found that just before the bow enters the lock, vessel with speeds 2 to 4 times higher than the limiting speed in the lock generate huge translatory waves in the lock. Vrijburcht (2000) describes the translatory waves generated during the lock entry and departure manoeuvre and proposes an equation for the maximum speed when the bow of the vessel enters the lock.

An overview of the longitudinal dynamics of ship entry and departure at locks has been presented by Spitzer and Söhngen (2013). They summarise investigations in Germany and other countries including numerical solutions of ship dynamics and field and model measurement data. In this paper the vessel speed in the lock is compared to the limiting speed according to Schijf's method.

The Panama Canal Commission performed prototype measurements of vessel speeds in the Miraflores upper west lock (Povirk and Rush,1999). In the Miraflores lock the longitudinal culverts are used for the return flow of the vessel to increase the vessel speed in the lock.

1.5 Desk study and prototype measurements

The present paper focusses on the vessel speed in the lock. The lock entry speed has been studied with the method described by Schijf (1949) and Jansen & Schijf (1953) as described in Section 2.2. According to this approach the possible speed of a vessel is a function of the blockage ratio and the water depth in the lock (see Section 2.1 for definitions). Where sailing in a canal can be characterised as sailing in confined water with a blockage ratio of: $0.1 < k < 0.2$, sailing in a lock should be characterised as sailing in extremely confined water with blockage ratio of $k > 0.5$.

From the desk study (RWS-ON,2016) and the calculation of the vessel speed the following can be concluded:

- The time required for a loaded Class Va vessel to enter the Lock Delden is significantly longer than for smaller vessels.
- If the maximum approach speed of 0.3 m/s (=1 km/hr) is respected, a loaded Class Va can be locked safely through the existing navigation Lock Delden.
- The vessel speed that is judged safe by skippers is significantly higher than the advised operational vessel speed (and even higher than the limited vessel speed computed with Schijf's method).

To verify the conclusions of the desk study, a verification study which incorporates prototype measurements near Lock Delden was executed. This finally resulted in a better understanding of the consequences of larger vessel size in the existing lock infrastructure, and an method for the accurate calculation of vessel speeds for a vessel in locks with high blockage ratio.

2 Definitions and calculation methods

In Section 2 the basic definitions and calculation methods used in this paper are presented:

1. Definition of the clearances and clearance ratios between the lock and the vessel;
2. Schijf's method for the calculation of the vessel speed in the lock; and
3. Determination of maximum vessel speed for safe entry of locks according to skippers.

Modifications on Schijf's method are presented in Section 5.1

2.1 Definitions of clearances and blockage ratio

The clearances between lock and vessel are related to the size of the vessel, whereas the blockage ratio is related to the wet cross-section of the lock, as defined below:

Length clearance: $LC = L_{lock} - L$, and length clearance ratio: $k_{LC} = LC/L$, with:

L_{lock} = Length of lock [m]
L = Length of vessel [m]

Beam clearance: $BC = W_{lock} - B$, and beam clearance ratio: $k_{BC} = BC/B$, with:

W_{lock} = Width of lock [m]
B = Beam of vessel [m]

Under-keel clearance: $UKC = D_{lock} - T$, and under-keel clearance ratio: $k_{ukc} = UKC/T$, with:

D_{lock} = Water depth in lock [m]
T = Draft of vessel [m]

Blockage ratio: $k = A_s / A_{lock}$ with:

A_{lock} = Wet cross-section of the undisturbed lock ($A_{lock} = W_{lock} \cdot D_{lock}$)
A_s = Wet cross-section of the vessel ($A_s = B \cdot T$)

2.2 Schijf's method for a Lock

At the bow of a sailing vessel water is constantly pushed aside, while at the same time an equal discharge of water is supplemented behind the vessel at the stern. This displacement of water from the bow to the stern induces a flow along and under the vessel, called the return flow. The return flow induces a water level depression around the vessel which results in a sinkage of the vessel compared to the still water level.

The speed of the vessel is limited by the return flow around the vessel hull. As the return flow increases (i.e. speed of the vessel increases), the water level depression increases until it reaches its maximum related to Froude. When this critical water level depression is reached, further increase in discharge of the return flow is not possible. This is a physical limit on the maximum speed of a vessel in the lock.

A vessel exceeding the limiting speed in the lock entrance creates a high translatory wave (Vrijburcht,1994). This wave, after reflection at the dead end of the lock and back at the bow significantly decelerates the vessel. Prototype measurements in the Lüneburg ship lift indicated that the average speed of the vessel in the lock is almost independent of the approach speed (Spitzer and Söhngen, 2013). This because of the physical limitation on the maximum discharge in the return flow.

The limiting speed (or critical speed) can be computed from the following equations originally derived by Schijf (1949) and Jansen and Schijf (1953). The equations have been derived for a vessel with a prismatic amidships cross-section over the total length of the vessel, sailing at constant speed and in a straight and prismatic canal section. The limiting vessel speed V_{lim} can be iterated from:

$$\frac{V_{lim}}{\sqrt{gD_{lock}}} = \left(\frac{2}{3}\right)^{3/2} \left\{1 - \frac{A_s}{A_{lock}} + \frac{1}{2}\left(\frac{V_{lim}}{\sqrt{gD_{lock}}}\right)^2\right\}^{3/2} \tag{1}$$

With:

V_{lim} = limiting vessel speed in the lock [m/s]
g = acceleration of gravity [m/s^2]

The corresponding return flow U_{lim} (m/s) around the vessels hull at limiting speed:

$$\frac{U_{lim}}{\sqrt{(gD_{lock})}} = \sqrt{\frac{2}{3}\left(1 - \frac{A_s}{A_{lock}} + \frac{1}{2}\left(\frac{V_{lim}}{\sqrt{(gD_{lock})}}\right)^2\right)} - \left(\frac{V_{lim}}{\sqrt{(gD_{lock})}}\right) = \left(\frac{V_{lim}}{\sqrt{(gD_{lock})}}\right)^3 - \left(\frac{V_{lim}}{\sqrt{(gD_{lock})}}\right) \tag{2}$$

The limiting water-level depression z_{lim} around the vessel's hull:

$$\frac{z_{lim}}{D_{lock}} = \frac{1}{3}\left(1 - \frac{A_s}{A_{lock}} - \left(\frac{V_{lim}}{\sqrt{(gD_{lock})}}\right)^2\right) \tag{3}$$

Sailing a vessel at a speed close to V_{lim} is not very realistic. In practice most of the vessels will not go much faster than about 80 % of this limiting speed due to the high power required and the associated fuel consumption for a further increase of speed (PIANC,2015). A more realistic maximum vessel speed is often set at 85% of the limiting speed. For more realistic sailing speeds $V < V_{lim}$ the return flow U can be iterated from:

$$\frac{\alpha(V+U)^2-V^2}{2gD_{lock}} - \frac{U}{V+U} + \frac{A_S}{A_{lock}} = 0 \qquad (4)$$

With:

- U = return flow in the lock [m/s]
- V = vessel speed in the lock [m/s]
- α = Correction factor for non-uniform distribution of the return flow ($\alpha = 1.4 - 0.4\frac{V}{V_{lim}}$)

And the water-level depression z from:

$$z = \frac{\alpha(V+U)^2-V^2}{2g} - \frac{V^2}{2g} \qquad (5)$$

The above method has initial been applied for calculation of the vessel speed in the locks.

2.3 Safe vessel entry speed

The operational vessel speed is often set at 85% of the limiting speed. However, a German study (VBD, 1993) has included the judgement of pilots to analyse the safety of the lock entering manoeuvre. It was found that a safe entry is determined by factors such as the approach speed, the draft ratio ($D_{lock}/T = 1 + k_{UKC}$) and the area ratio ($n = 1/k$).

These studies have been re-evaluated for the UKC ratio: $0.07 \leq k_{UKC} \leq 0.60$ and blockage ratio: $0.595 \leq k \leq 0.885$ which led to the results shown in Figure 2 (BAW,2005).

The graph presents an estimate for the safe entry speed ratio ($= v_{safe}/\sqrt{g \cdot D_{lock}}$) as function of the under-keel clearance ratio k_{UKC} and the beam ratio k_{BC}.

Note that in Figure 2 the definitions of the ratios have been adopted to the ratios defined in Section 2.1 of this paper.

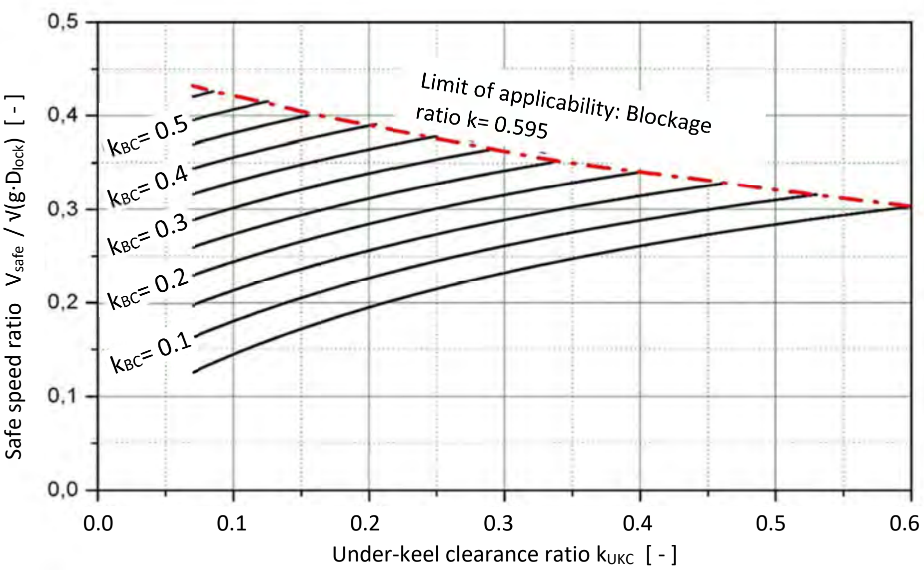

Figure 2: Safe speed at lock entry manoeuvre (BAW,2005)

The Class IV and Class Va vessels in Table 1 have a blockage ratio in between the limits of applicability of Figure 2. Therefore, Figure 2 can be used to determine the safe entry manoeuvre speed for these vessels in Lock Delden. The safe speed for the vessels entering Lock Delden (RWS-ON, 2016) turned out to be about 3 to 4 times higher than the limiting speed calculated with Schijf's method. From this comparison it turns out that both the limiting and the operational vessel speed are considered to be safe.

As mentioned in Section 2.2, if the vessel enters the lock with the speed that is considered safe, but higher that the limited speed, the vessel will induce a wave in the lock chamber that runs through the lock and will be reflected at the dead end of the lock. Once the reflected wave returns at the bow of the vessel, the vessel will be slowed down significantly. To avoid these fluctuations in the vessel speed the skipper is advised to reduce his speed in the approach to the lock to less than the limiting speed in the lock.

3 PROTOTYPE MEASUREMENTS

3.1 Objective of the measurements

The objective of the prototype measurements is to measure the impact of the increase of the maximum draft for a Class Va from 2.20 m to 2.80 m on the locking process in Lock Delden. To get prototype data for a range of blockage ratios and limit the risk of grounding the vessel, the prototype measurements were executed for two drafts. The first measurements were done with draft of 2.6 m (original design depth of the channel) and finally with the future maximum of 2.8 m. During the prototype test, the following information has been collected from the prototype vessel:

- Position and speed of the vessel during normal navigation in the canal before, during and after the locking process, including entering/departure in the lock and the locking itself.
- The under-keel clearance in the lock head and above the sill and in the lock;
- The lock-cycle time.

In addition, the prototype dimensions of the lock have been measured accurately in situ.

3.2 Lock Delden

Lock Delden is situated in the Twentekanaal 36 km from the junction with the IJssel River. The lock heads have been made of concrete and the lock chamber is created with sheet pile walls. Lock Delden has two lift gates between towers pairs. The main dimensions of the lock are presented in Table 2:

Dimension of lock		
Length	lock structure	159 m
	lock chamber between gates	140 m
	utilised length for vessels	133 m
Width		12.05 m
lock lift		6.00 m

Levels and water depth		Lower lock head
Water level	normal	NAP +10.00 m
	minimum	NAP + 9.90 m
Sill level		NAP + 6.56 m
Minimum water depth lower lock head		3.34 m

Levels and water depth		Upper lock head
Water level	normal	NAP +16.00 m
	minimum	NAP + 15.90 m
Sill level		NAP + 12.25 m
Minimum water depth upper lock head		3.65 m

Table 2: Lock dimensions and water depths

Figure 3: Lock Delden from West to East

3.3 Measurement plan

The Twentekanaal from Lock Delden up to Hengelo has been presented in Figure 4. Lock Delden (Km 36,32) is approximately 2 km from the junction with the branch to Almelo (Km 32) and approximately 8 km from the CTT (Km 44). The water bed bottom in this 10 km section of the canal and the lock has been surveyed beforehand to verify the water depth.

Figure 4 Twentekanaal: section for prototype test

During the night the traffic on the canal is low, especially in the weekend. During one night the Class Va container vessel could make two roundtrips passing the lock four times. Therefore, the prototype test was scheduled for a night in the weekend: Friday 31 March up to Saturday morning 1 April 2017. The vessel turned near the Combi Terminal Twente (CTT) and at the junction with the canal branch to Almelo. The vessel is loaded with containers to the required draft at the CTT in Hengelo and sailed up and down the Twentekanaal (Figure 4) once with a draft of 2.6 m and once with a draft of 2.8 m

3.4 Prototype vessel characteristics

The prototype vessel is a Class Va container vessel that has been scheduled on a roundtrip between the Port of Rotterdam and the CTT in Hengelo on a regular basis since 2008. The skipper of the vessel is therefore very familiar with the Twentekanaal. The main characteristics of the prototype vessel are[4]:

Main characteristics of the vessel		
Year of construction		2006
Hull		bulk / containers
Length		110 m
Beam		11.45 m
Maximum draft		3.50 m
Capacity	cargo	3080 Ton
	containers	208 TEU
Main engine		Caterpillar 1699 Hp (1249 kW)
Bow thruster		2 * DAF 430 Hp (2 * 316 kW)

Table 3: Vessel characteristics

Figure 5: Prototype vessel with containers

3.5 Instruments and data registration

The vessel has been equipped with instruments to register the manoeuvring actions of the skipper, and to measure the position of the vessel. The manoeuvring actions have been captured in the wheel house by video. The video registration (see Figure 6) has been analysed with video capturing software to create time series of the registration. Recordings have been made of:

1. Main engine: rpm, load factor and the fuel consumption;
2. Ruder: ruder angle; and
3. Bow thruster: rpm and direction.

[4] Source.: http://www.binnenvaartcruises.nl/schepen/martinique and https://www.binnenvaart.eu/motorvrachtschip/3057-anne-marijke.html .

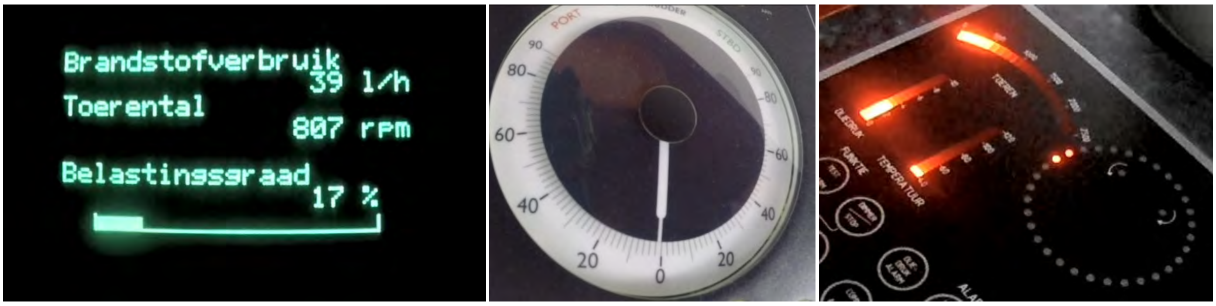

Figure 6: Video registration of meters of main engine, ruder and bow thruster

The position of the vessel has been permanently measured by two sets of GPS receivers (RTK heading receiver). One set has been installed at the bow and one at the stern with 102.5 m between each other. The two receivers have been placed near the centre-line of the vessel, around 6 m above the water level close to the top level of the containers. The GPS measurements (x, y, z and heading) have been stored with a frequency of 1 Hz.

In each lock head two pressure transducers have been installed, one on each side of the lock gate. The transducers have been installed in the sheet pile wall along the south side of the lock, about one meter below the lowest water level. The transducers measure the water pressure (1 Hz) and thereby the water level. Examples of the resulting time series are presented in Section 3.6.1 to 3.6.3.

3.6 Prototype measurement and collected data

The prototype measurements have been performed according to the four tracks of the measurement:

Track 1: Sailing and descending with a draft of 2.6 m
Track 2: Sailing and ascending with a draft of 2.6 m
Track 3: Sailing and descending with a draft of 2.8 m
Track 4: Sailing and ascending with a draft of 2.8 m

During the evening of Friday 31 March and the night of 1 April the traffic on the canal had become quiet and the water level had come to rest. The skipper sailed the tracks with the main engine on slow or dead-slow. The skipper did not experience any specific problem and after the measurements he proceeded his schedule to Nijmegen/Rotterdam.

The lock entry speed was much lower than the safe lock entry speed derived in Section 2.3. The manoeuvres have been characterised as safe, and the vessel did not touch the lock or the lock head unintentionally. The next sections show examples of the registration of the manoeuvring actions of the skipper, the vessel speed and the water level registrations (Aktis,2017).

3.6.1 Data from the main engine ruder and bow thruster

An example of the resulting timeseries of the main engine has been presented in Figure 7.

The graphs show the data from the main engine (Dutch: ***hoofdmotor***): the revolutions (Dutch: ***Toeren***), the fuel consumption (Dutch: ***Brandstofverbruik***) and the load factor (Dutch: ***Belastingsgraad***).

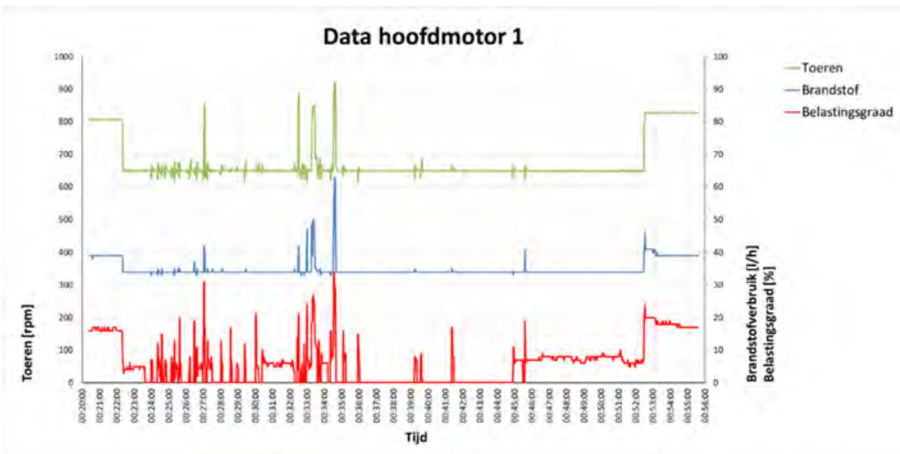

Figure 7: Main engine during sailing and ascending up with a draft of 2.6 m (Track 1)

The vessel entered the lock between 00:30:34 (bow in lock head) and 00:32:57 (stern in lock head) and Figure 7 indicates that during the entry manoeuvre the vessel sailed with the engine on dead-slow. The period in between 00:37:30 and 00:44:20 hours the lock was levelled, which is shown by the load factor of practically zero. The bow left the lock at 00:47:40 and the stern left the lock at 00:52:29. As the stern left the lock the load factor increases.

3.6.2 Data from the GPS: vessel speed at bow and stern (aft)

The timeseries from the GPS have been analysed and processed. The speed of the middle of the vessel was calculated by averaging the speed of the bow and the stern. An example of the resulting timeseries of the computed vessel speed has been presented in Figure 8.

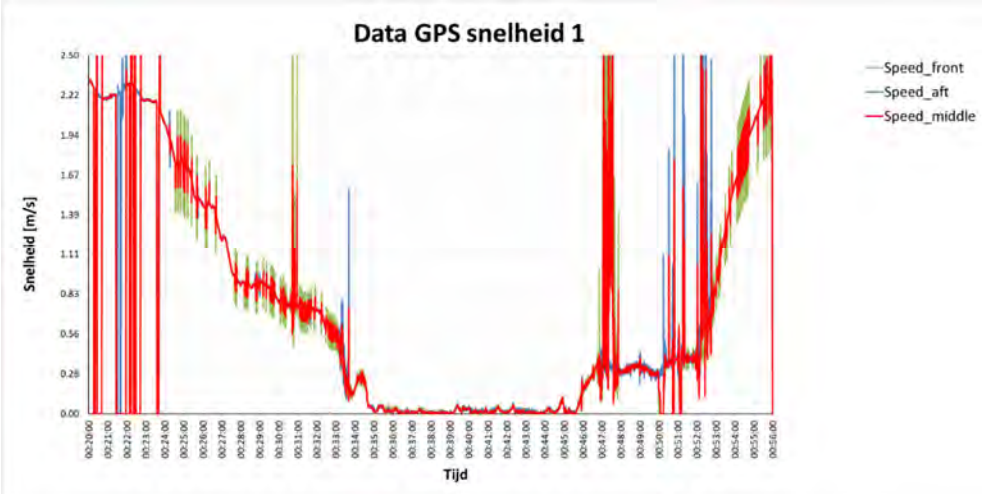

Figure 8: Vessel speed from GPS during sailing and descending with a draft of 2.6 m (Track 1)

The graph shows the vessel speed data (Dutch: ***Snelheid***) from the GPS. From 00:30:34 to 00:37:30 and from 00:44:20 to 00:52:29 hours the vessel was in a lock head. These periods correspond to a grassy speed signal as the GPS signal was disturbed by the presence of the lock gates. Fortunately, only one GPS at a time was disturbed. The figure below shows the vessel speed (Dutch: ***Snelheid***) and squat as function of the position of the vessels bow and stern (Dutch: respectively: ***boeg*** and ***hek***) during the entering of the lock.

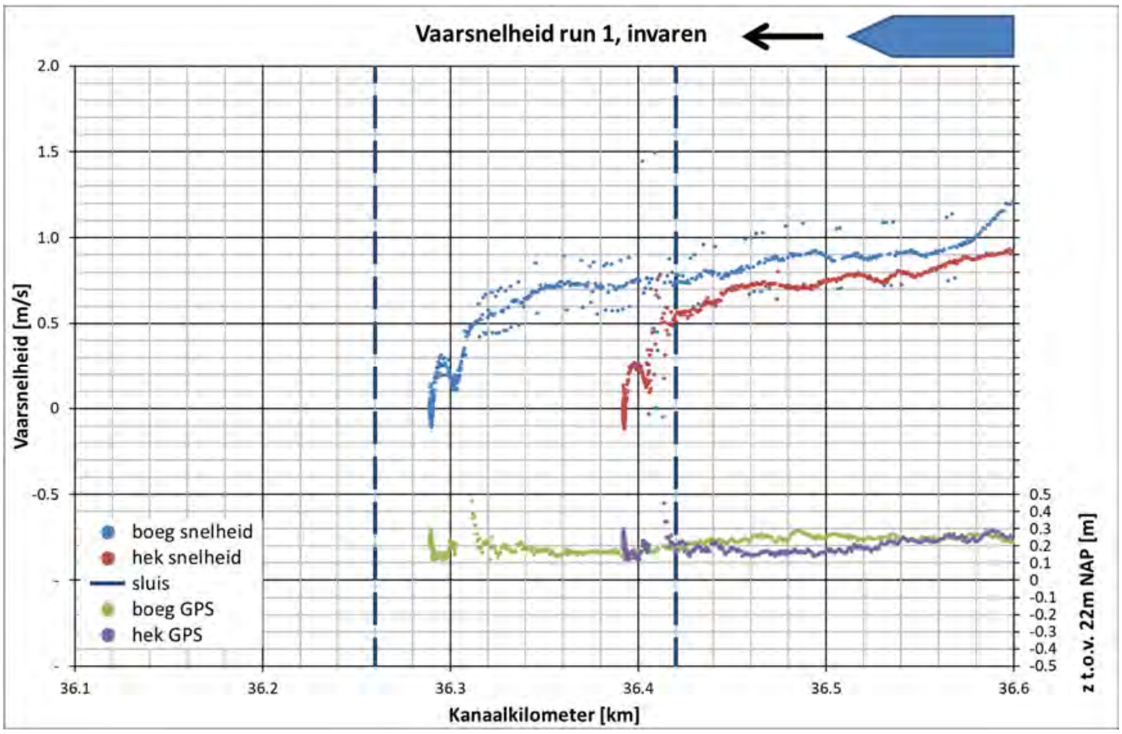

Figure 9: Vessel speed and squat w.r.t. position in the lock with a draft of 2.6 m (Track 1)

3.6.3 Data from the pressure transducers: water level in the canal and in the lock

The timeseries from the pressure transducers have been analysed and processed. An example of the resulting timeseries of the water level has been presented in Figure 10.

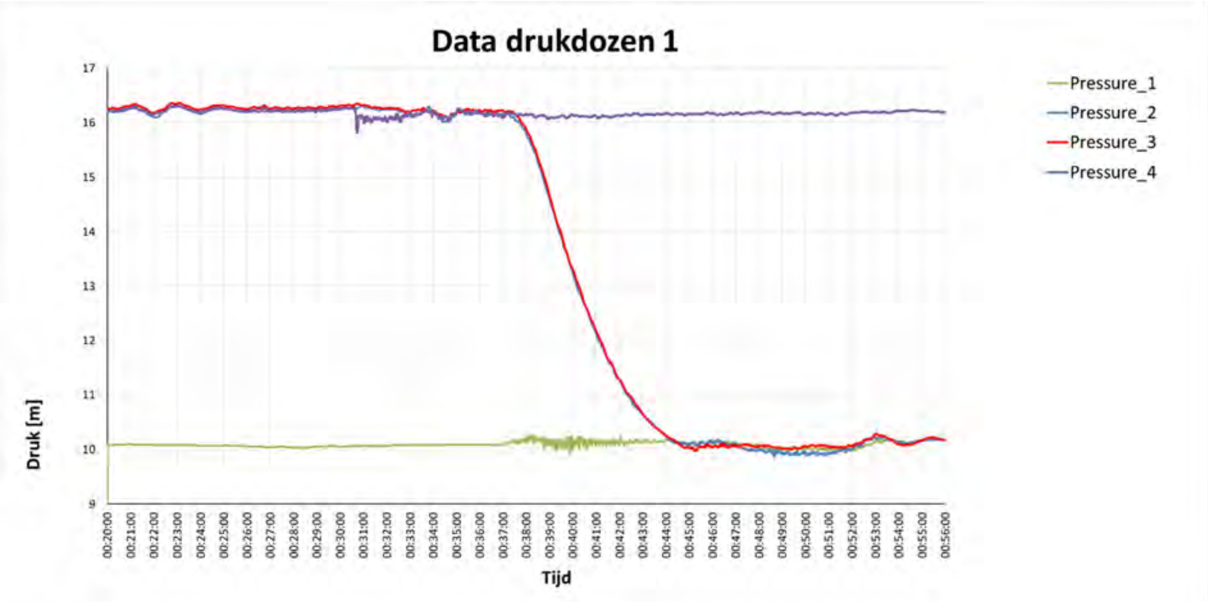

Figure 10: Water level near the lock gates: descending with a draft of 2.6 m (Track 1)

The graphs show the water pressure (Dutch: ***Druk***) from the pressure transducers (Dutch: ***drukdozen***) as measure for the water level at both sides of the lock gates. From 00:30:34 to 00:52:29 hours the vessel was in the lock. At the start of this period the water level in the upper lock head (Pressure 4) and in the lock chamber (3 and 2) are disturbed by the entering vessel. Subsequently, the lock chamber has been levelled (Pressure 2 and 3). During the departure of the vessel from the lock chamber small water level variations can be seen in the lower lock head (Pressure 2 and 1).

3.7 Input for verification of Schijf's method

The following values have been derived for the four sailing tracks for the understanding of the hydrodynamic process:

- Dimensions of the lock and the water level;
- Use of main engine (propeller) during lock entry and lock departure manoeuvre;
- Sailing speed in the lock during sailing in and during sailing out;
- Maximum squat of bow and stern in lock head;
- Drop in water pressure in the lock head during passing of vessel;
- Duration of lock entering (from bow in lock head to stern in lock head); and
- Duration of lock departure (from bow out of lock head to stern out of lock head).

The verification of the vessel speed with Schijf's method requires the following representative values as input:

1. the lock width and water depth in lock (above the sill); and
2. the beam and draft of the vessel;

and for verification:

3. the average speed of the vessel during entering and departing the lock; and
4. the water level depression and squat of bow and stern passing the lock head.

3.7.1 Vessel speed in the prototype test

The vessel speed in the lock varies slowly (see Figure 9). The average vessel speed has been determined from the duration of the entry or departure manoeuvre and the corresponding sailing distance. The duration has been defined as the time between the bow and the stern reaching the same lock head. The measured average vessel speed for each of the four entry and departure manoeuvres is presented in Table 4.

Sailing			Lock entry		Lock departure		Lock passing time	
Track	Direction	draft of vessel	Duration (102.5m)	average speed	Duration (102.5m)	average speed	from bow in lockhead till stern through lock head	
			[s]	[m/s]	[s]	[m/s]	[s]	[min]
1	down	2.6 m	143	0.717	289	0.355	1315	21.9
2	up		262	0.391	162	0.633	1543	25.7
3	down	2.8 m	178	0.576	392	0.261	1434	23.9
4	up		374	0.274	201	0.510	1721	28.7

Table 4: Average vessel speed during lock entry and departure and lock passing time

The numbers in Table 4 clearly indicate that the 0.2 m extra draft has a significant effect on the duration of the lock entry and departure manoeuvre (25-40% longer).

3.7.2 Water depth and under-keel clearance on the sills during the passing of the vessel

The vertical position of the bow and the stern of the vessel have been measured with the GPS receivers. The main interest was the squat and the remaining under-keel clearance in the lock head above the sill. Unfortunately, the lifted lock gates hinder the GPS signal. Consequently, there are some gaps in the time series of the vertical position of the bow and stern in the lock heads. However, from the GPS signal during the lock approach and in the lock itself, an indication of the squat in the lock head could be derived.

The pressure transducers provide information on the water level depression in the lock head. In addition the actual water levels in the lock heads have been derived from the pressure transducers. During the passing of the vessel the water level fluctuated a few centimetres up to (very short peak of) five decimetres. The largest depressions were measured around the bow during the entry of the bow into the lock head. The representative water level depression and the squat in the lock head during the entry and departure manoeuvres are combined in Table 5.

Sailing		Measurement in lock head					Under keel clearance	
Track	Direction	sill level	water level	draft of vessel	squat from GPS	water level lowering	UKC from GPS	UKC from water level
		NAP+ m	NAP+ m	[m]	[m]	[m]	[m]	[m]
Lock entry manoeuvres:								
1	down	12.25	16.15	2.609	-0.15	-0.45	1.14	0.84
2	up	6.56	10.00		0.05	-0.15	0.88	0.68
3	down	12.25	16.10	2.782	-0.10	-0.40	0.97	0.67
4	up	6.56	10.00		0.05	-0.20	0.71	0.46
Lock departure manoeuvres:								
1	down	6.56	10.05	2.609	-0.20	-0.20	0.68	0.68
2	up	12.25	16.10		-0.15	-0.15	1.09	1.09
3	down	6.56	10.00	2.782	-0.15	-0.20	0.51	0.46
4	up	12.25	16.05		-0.15	-0.10	0.87	0.92

Table 5: Squat, water depth and under keel clearance in the lock heads on the sill

The numbers in Table 5 indicate a significant difference between the UKC estimated from the GPS measurements and the UKC based on the water level from the pressure transducers. The lowest UKC values are found in the lower lock head during the departure manoeuvres. The lock is far from being touched, since the UKC is 0.45 m or more.

4 VERIFICATION OF SCHIJF'S ORIGINAL METHOD

4.1 Computations for verification of Schijf's method for vessel speed in locks

Schijf's method as presented in Section 2.2 has been applied to calculate the vessel speed for the water depth and draft conditions that occurred during the measurements. The entering conditions, the measured entering time, the average vessel speed (see Section 3.7.1), and the calculated vessel speed for the measured entering and departure conditions are presented in Table 6.

Verification and Calibration			Track 1		Track 2		Track 3		Track 4	
sailing distance 102.5		m	upper-in	lower-out	lower-in	upper-out	upper-in	lower-out	lower-in	upper-out
Lock Delden	Width	[m]	12.05	12.05	12.05	12.05	12.05	12.05	12.05	12.05
	water depth in lock head	[m]	3.85	3.49	3.49	3.85	3.85	3.49	3.49	3.85
Vessel	Beam	[m]	11.40	11.40	11.40	11.40	11.40	11.40	11.40	11.40
	Draft	[m]	2.609	2.609	2.609	2.609	2.782	2.782	2.782	2.782
Blocking ratio		[%]	64%	71%	71%	64%	68%	75%	75%	68%
	Sailing time (over 102.5 m)	[s]	143	289	262	162	178	392	374	201
	Delden measured vessel speed	[m/s]	0.72	0.35	0.39	0.63	0.58	0.26	0.27	0.51
Schijf's method	v_{op}	[m/s]	0.63	0.44	0.44	0.63	0.52	0.33	0.33	0.52
	Error w.r.t. measurment	[%]	-12.1%	23.3%	11.8%	-0.4%	-10.1%	28.0%	22.1%	1.5%

Table 6: Measured and calculated vessel speed

The measured and calculated sailing speed and the time for sailing in and out of the lock deviate significantly and systematically. The results from Schijf's method indicate that:

- The computed vessel speeds differ up to 28% from the measured vessel speed;
- The vessel speeds computed for the upper lock head are too low;
- The vessel speeds computed for the lower lock head are too high;
- Departing takes more time than entering the same lock head.

The above findings indicate that the agreement between the measured and the calculated vessel speeds can be improved by the adaption of Schijf's method for the systematic deviations.

5 CALIBRATION OF SCHIJF'S METHOD

5.1 Schijf's method for vessel speed in locks

An explanation for the systematic deviation between the measured and the calculated vessel speeds are the hydraulic processes in the lock which are not accounted for in Schijf's method:

- The effect of the dead-end waterway, resulting in increased water level in the lock in front of the bow;
- Hydraulic resistance between vessel hull and lock chamber that results in a water level inclination along the vessel.

The above mentioned two effects have been added to Schijf's original method by a correction of the water depth in the lock head. The corrections are elaborated in the next section.

5.1.1 Adaption for dead end lock

A vessel entering a lock with speed below the limiting speed increases the water level in the dead of the lock. The water level in the return flow in the lock is lower than in the dead end of the lock. This operational water level depression (z_{op}) in the return flow can be computed with Schijf's method. In the adaptation of Schijf's method it is assumed that the water level in the return flow of the vessel is equal to the water level in the outer harbour. This implies that the water level at the bow in the dead-end of the lock is z_{op} higher than in the return flow around the vessel and thus z_{op} higher than in the canal.

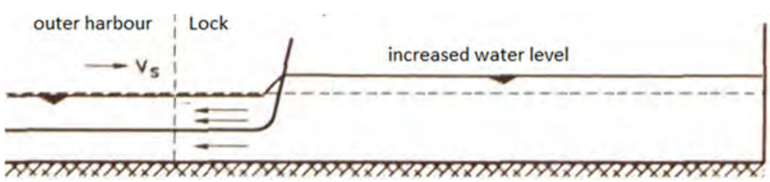

Figure 11: Water level during lock entry

At the beginning of the lock entry manoeuvre when the bow just enters, the vessel is hardly affected by the higher water level in the lock. However close to the end of the entry manoeuvre the vessel is lifted z_{op} above the still water level assumed in the calculation. It is assumed that on average a vessel entering a lock is lifted 50% of the calculated z_{op} above the still water level in the canal.

The flow around a vessel departing from a lock is similar to sailing in a canal, with a water level around bow and stern equal to the still water level. As such the effect of a dead end-lock is not present during a lock departure manoeuvre.

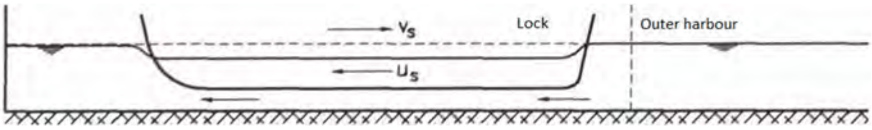

Figure 12: Water level during lock departure

The following adaption ΔD_1 is made for the dead-end effect:

$$\Delta D_1 = C_1 \cdot z_{op} \tag{6}$$

With:

- z_{op} = Operational water level depression around the vessel in the lock [m],
- C_1 = Factor based on the manoeuvre: $C_1 = 0.5$ for lock entry; $C_1 = 0$ for lock departure [-].

5.1.2 Adaption for hydraulic resistance in return flow around the vessel

In the situation with a high blockage ratio the clearance between the vessel and the lock is very small, while at the same time the return flow velocity is extremely high. This combination results in a significant hydraulic resistance and therefore a water level slope in the return flow. lock that affects the water depth in the lock. Therefore, it has been proposed to adapt the hydraulic resistance in the computation. The water level slope i [m/m] in the return flow can be calculated following Chézy:

$$i = \frac{v^2}{C^2 * R} \tag{7}$$

With:

- v = Return flow around the vessel [m/s]
- R = Hydraulic radius of the wet area between the vessel and the lock wall [m]:

$$R = (D_{lock} \cdot W_{lock} - T \cdot B)/(2 \cdot D_{lock} + W_{lock} + 2 \cdot T + B) \tag{8}$$

- C = Coefficient of Chézy [$m^{1/2}/s$]:

$$C = 18 * \log(\frac{12 * R}{k_N}), \tag{9}$$

With:

$k_N = 0{,}001$ (Nikuradse roughness representative for surface of vessel and lock)

De Nikuradse roughness ($k_N = 0{,}001$) has been selected to be representative for the steel and concrete surface of the vessel and Lock Delden.

When sailing through the lower head the water level slope is present over the whole length of the vessel. Sailing through the upper head of the lock the water level slope is only applicable in the shallow part in the lock head. The length of the lock head in Lock Delden is estimated at some 20% of the length of the vessel. It is proposed to adapt the water depth in the computation of the vessel speed with the water level slope over the length of the vessel (lower head) or the length of the lock head (upper head). The coefficient ${C_B}^2$ has been added to account for the amidships cross-section being present over part of the vessel length. This results in the following adaption ΔD_2 for the slope of the water level in the return flow:

$$\Delta D_2 = C_2 \cdot -i \cdot L \cdot {C_B}^2 \tag{10}$$

With:

- C_2 = Factor depends on the lock head: $C_2 = 1.0$ lower; $C_2 = 0.2$ upper lock head [-].
- L = Length of vessel [m]
- C_B = Block coefficient of vessel: $C_B = 0.9$ for tested vessel [-]

5.1.3 Combining the adaptations

The corrected water depth in the lock due to the proposed adaptions are combined in:

$$D_{lock,adapted} = D_{lock} + C_1 \cdot z_{op} + C_2 \cdot -i \cdot L \cdot {C_B}^2 \tag{11}$$

The proposed values for C_1 and C_2 are summarized in Table 7:

Sailing direction	Lock head	Dead-end ahead of the bow	Water level slope in return flow
Lock entry	Lower	C_1 = 0.5	C_2 = 1.0
Lock entry	Upper	C_1 = 0.5	C_2 = 0.2
Lock departure	Lower	C_1 = 0.0	C_2 = 1.0
Lock departure	Upper	C_1 = 0.0	C_2 = 0.2

Table 7: Proposed coefficients in adaptation of Schijf's method

The above proposed adaptions have been added in Schijf's method via an iterative calculation process. In the first iteration the vessel speed, water level depression and return flow are calculated with the original method. The water level depression and return flow are then applied to calculate the adaption to the water depth in the lock. In the subsequent steps the water level adopted from the previous computational step is used as input for Schijf's method.

5.2 Computations for calibration of Schijf's method adapted for vessels in locks

The calculations with the adaptation of Schijf's method have been executed for the conditions that occurred during the measurements. The main results are presented in Table 8 together with the measured vessel speed and the results from Schijf's original method.

Verification and Calibration		Track 1		Track 2		Track 3		Track 4	
sailing distance 102.5	m	upper-in	lower-out	lower-in	upper-out	upper-in	lower-out	lower-in	upper-out
Lock Delden Width	[m]	12.05	12.05	12.05	12.05	12.05	12.05	12.05	12.05
water depth in lock head	[m]	3.85	3.49	3.49	3.85	3.85	3.49	3.49	3.85
Vessel Beam	[m]	11.40	11.40	11.40	11.40	11.40	11.40	11.40	11.40
Draft	[m]	2.609	2.609	2.609	2.609	2.782	2.782	2.782	2.782
Blocking ratio	[%]	64%	71%	71%	64%	68%	75%	75%	68%
Sailing time (over 102.5 m)	[s]	143	289	262	162	178	392	374	201
Delden measured vessel speed	[m/s]	0.72	0.35	0.39	0.63	0.58	0.26	0.27	0.51
Schijf's method v_{op}	[m/s]	0.63	0.44	0.44	0.63	0.52	0.33	0.33	0.52
Error w.r.t. measurment	[%]	-12.1%	23.3%	11.8%	-0.4%	-10.1%	28.0%	22.1%	1.5%
Schijf+Adaption v_{op}	[m/s]	0.68	0.36	0.40	0.62	0.56	0.25	0.28	0.50
Error w.r.t. measurment	[%]	-5.2%	1.9%	3.4%	-2.3%	-3.2%	-4.7%	3.0%	-1.0%

Table 8: Measured and calculated vessel speed without with adapted method

The computed values are plotted w.r.t. the measured values in Figure 13.The error in the computed values w.r.t. the measurements are in most cases significantly smaller for the adapted method.

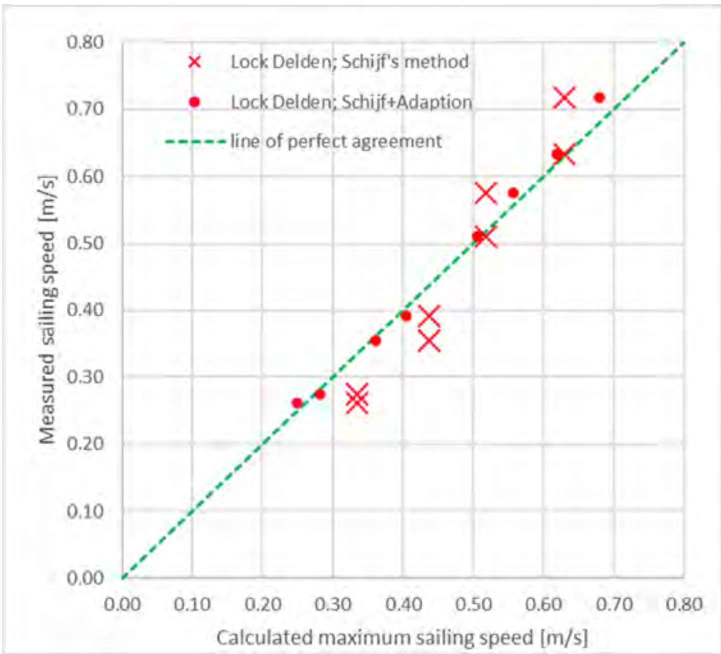

Figure 13: Measured and calculated vessel speed without and with adapted method

5.3 Conclusions on the verification with data from prototype measurements

Concerning the proposed adaption of Schijf's method for the calculation of the vessel speed in locks the following conclusions were drawn.

- With the proposed adaption of Schijf's method the differences between the measured and calculates lock entry and departure speed is 5% or less.
- The difference between the computed and measured velocities reduced from a maximum of 28% for Schijf's original method to 5% or less with the adapted method.
- The validation covered blockage ratios between 64% and 75% ($0.64 < k < 0.75$).

The conclusions on the adaptation of Schijf's method are based on 8 prototype measurements from Lock Delden. In the next section prototype measurements from other locks and from a ship lift are added to the data set for validation of the adaptation of Schijf's method.

6 COMPARISONS WITH DATA FROM SAMBRE LOCKS AND LÜNEBURG LIFT

6.1 Prototype measurements on the Sambre

Prototype tests with a similar setup as for Lock Delden have been executed on the Sambre River in Belgium. The Sambre River is a meandering river in the southern part of Belgium. The river is part of the waterway connection between the Scheldt and the Meuse rivers. As part of the project Seine – Escaut Est (SEE) the navigation route will be increased from CEMT Class IV to CEMT Class Va vessels with a length of 110 m. In addition to several manoeuvring simulation studies, the Service Public de Wallonie (SPW) performed a prototype test to measure the behaviour of a large vessel navigating in the narrow and curved Sambre river (Bousmar e.a.,2013).

The SPW has reported the prototype measurements on the Sambre in a measurement report (SPW,2010). The measurements on the Sambre have been executed on 11 December 2009 with a loaded Class Va vessel with length: 105 m, beam: 10.5 m and draft: 2.6 m. The test covered sailing over 38 km of the Sambre river (from Pont-de-Loup up to Namur), including the passage of 5 navigations locks. The horizontal dimensions of the lock are $L_{lock} \geq 112$ m and $W_{lock} = 12.5$ m which complies with CEMT Class Va. During the passing of the locks the minimum water depth in the locks was $D_{lock} \geq 3.45$ m. The blockage ratio of the vessel was high ($0.46 < k < 0.63$) but not extremely high ($k > 0.75$) that special measures are required for locking the vessel (PIANC,2015).

The vessel was equipped with various instruments to register the manoeuvring action of the skipper and measure the position of the vessel. The water level depression in the Sambre was measured at various locations in the river during the passing of the vessel, but not in the lock. The type of prototype measurements and the recorded data correspond well with the prototype measurements for Lock Delden. This makes it possible to add the measurements from the Sambre to the present data set for the verification of the proposed adaption to Schijf's method.

6.2 Measurement and verification for the five Sambre locks

The measurements in the Sambre relate to sailing downstream. During the measurements the vessel the entered the lock through the upper head and the departure through the lower head. The vessel passed 5 unique locks with slightly different water depths in the lock heads. In the Sambre Locks E14 Auvelais and E17 Salzinnes the sills of the mitre gates are locally higher than the lock floor. These sills have hardly impact on the return flow, however, the higher level increase the risk of grounding the vessel at the sill at a higher sailing speed.

The measurements in the locks in the Sambre comprised the positions of bow and stern of the vessel, and the manoeuvring actions, but not the water levels in the lock heads. The position of the vessel was measured with three GPS antennas located at the bow, the stern and the centre point of the vessel that registered the position (X, Y and Z) of the vessel. The manoeuvring actions of the skipper have been registered by video cameras.

All signals have been verified and stored with a frequency of 1 Hz. Because the locks in the Sambre have horizontal moving (transversal) gates or mitre gates, the GPS measurements have not been interrupted by lifted lock gates (as was the case with the lifting gates in Delden). Additionally, the central GPS data enabled series reconstructions when missing data.

6.3 Verification calculations

The values from the prototype measurement have been collected for calculations. These data are presented in Table 9 together with the results of the calculations with Schijf's method.

Verification and Calibration		E13 Roselies		E14 Auvelais		E15 Mornimont		E16 Floriffoux		E17 Salzinnes	
sailing distance 93.6 m		upper-in	lower-out	upper-in	lower-out	upper-in	lower-out	upper-in	lower-out	upper-in	lower-out
Sambre Locks Length	[m]	112		136		112		112		136	
Width	[m]	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
water depth in lock head	[m]	3.84	3.95	4.16	3.45	4.70	4.39	3.92	4.64	4.45	3.85
Vessel Beam	[m]	10.50	10.50	10.50	10.50	10.50	10.50	10.50	10.50	10.50	10.50
Draft	[m]	2.60	2.60	2.60	2.60	2.60	2.60	2.60	2.60	2.60	2.60
Blocking ratio	[-]	0.57	0.55	0.53	0.63	0.46	0.50	0.56	0.47	0.49	0.57
Sailing time (over 93.6 m)	[s]	151	161	107	165	115	127	99	95	102	130
Measured vessel speed	[m/s]	0.62	0.58	0.87	0.57	0.81	0.74	0.95	0.99	0.92	0.72
Schijf's method v_{op}	[m/s]	0.84	0.90	1.02	0.62	1.32	1.15	0.89	1.29	1.18	0.85
Error w.r.t. measurment	[%]	36%	55%	17%	9%	62%	56%	-6%	31%	29%	18%
Schijf+Adaption v_{op}	[m/s]	0.91	0.86	1.10	0.56	1.42	1.12	0.96	1.26	1.27	0.80
Error w.r.t. measurment	[%]	46%	48%	26%	-1%	74%	51%	1%	28%	39%	11%

Table 9: Verification calculations of vessel speed for Sambre locks

The sailing speeds from Table 9 have been presented in Figure 14 (dark blue markers) together with the vessel speeds from Lock Delden (red markers). Figure 14 presents also some data from the Lüneburg ship lift (Light blue markers), which are further discussed in Section 6.4.

At first glance the data points in Figure 14 could give the impression that the measured and calculated vessel speeds for the Sambre locks are much less in agreement than for Lock Delden. Figure 14 indicates that the calculated vessel speeds in the Sambre locks are generally much higher than in Lock Delden. This corresponds to the fact that the blockage ratio in the Sambre locks ($0.46 < k < 0.63$) was in general lower than in Lock Delden ($0.64 < k < 0.75$), see also Figure 16.

The data points in Figure 14 indicate that the agreement is good in the lock heads where the (calculated) vessel speed is less than approximately 0.8 m/s (approximately 3 km/hr).

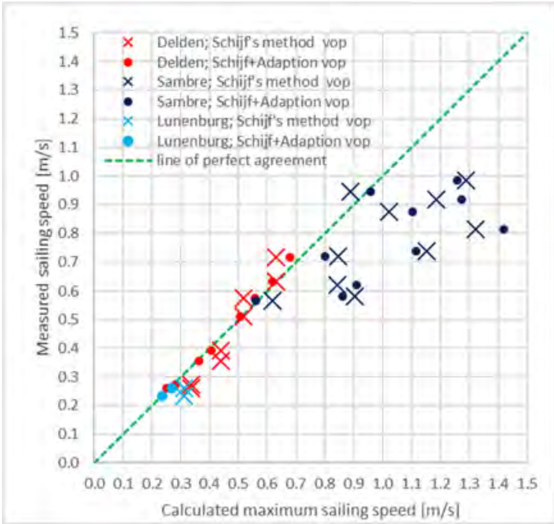

Figure 14: Measured and calculated vessel speed without and with adapted method

Figure 14 indicates that the calculated vessel speed forms an upper boundary for the vessel speed. The skipper intentionally sailed with a maximum speed of about 0.8 m/s (3 km/hr) even if that is much slower than physically possible.

Figure 15 presents the use of the main propeller during the entry manoeuvres in lock E13 and E15. The vertical lines mark the time that the bow and the stern enter the lock heads. The graphs indicate that in these Sambre locks the main engine was used only part-time during the entry manoeuvre (whereas in Lock Delden the main engine has been used throughout almost the entire entry manoeuvre, see Figure 7 for an example).

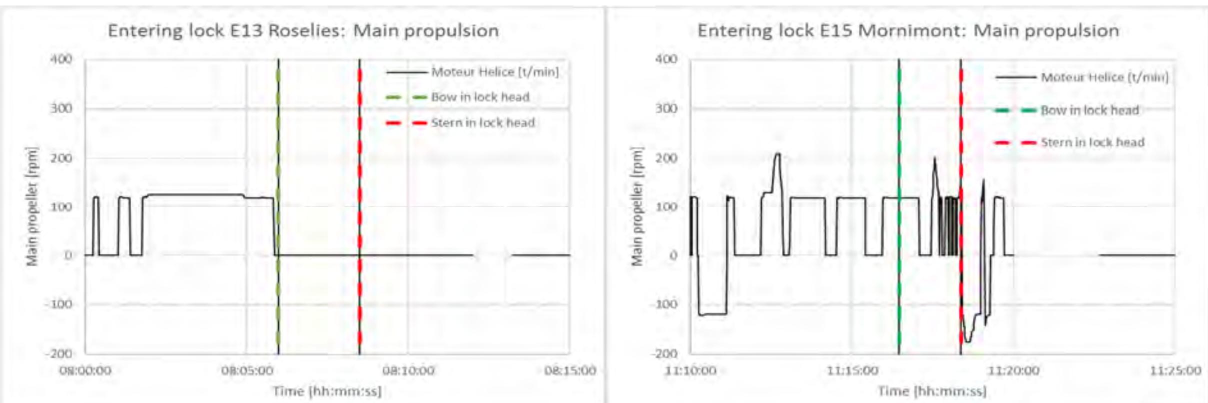

Figure 15: Entry manoeuvre Sambre locks; main propulsion and propeller

In Figure 16 the vessel speeds have been plotted as function of the blockage factor of the vessel in the respective lock heads. This presentation of the data indicates that the measured and calculated vessel speed are in good agreement for the blockage ratio higher than $k > 0.6$ and that for smaller blockage ratios the measured vessel speed is equal or lower than calculated. As such Schijf's method gives an accurate upper boundary for the vessel speed in locks with a low blockage ratio ($k < 0.6$).

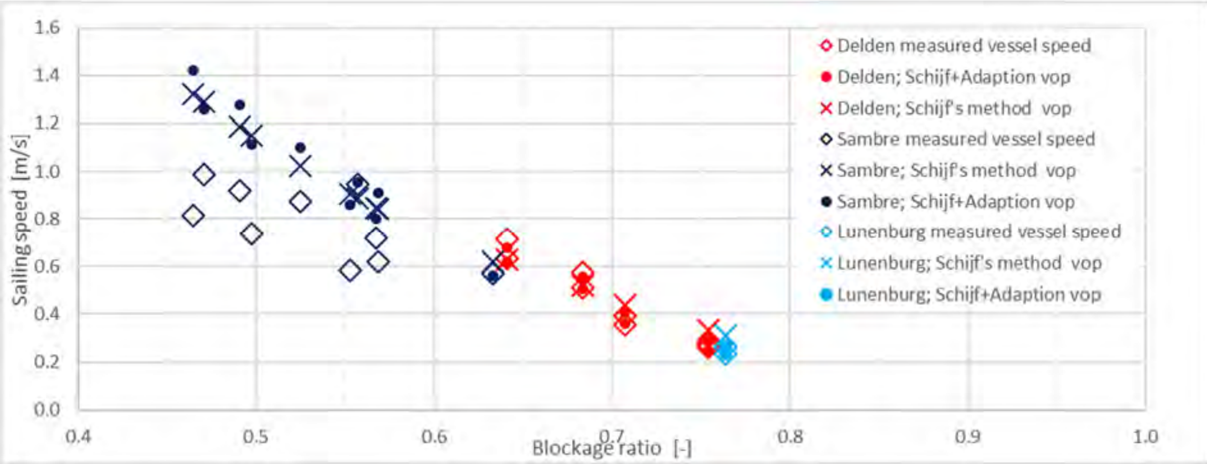

Figure 16: Measured and calculated vessel speed as function of blockage factor

6.4 Prototype measurement from the Lüneburg ship lift

Prototype measurements for the Lüneburg ship lift have been described by Spitzer and Söhngen (2013). The ship lift has dimensions of L=110 m, W=12.25 m and D=3.41 m. A total of 14 measurements were carried out. The measurements comprised the entering of the lift and the departure from the lift. The vessel entered the ship lift with an approach speed that varied from 2 to 8 km/hrs, whereas the blockage ratio was very high ($k > 0.75$), which results in a limiting speed in the ship lift of 0.37 m/s (1.3 km/hrs).

The average vessel speed in the ship lift has been adopted from the graph and presented in Table 10. The results have also been presented in the Figure 14 and Figure 16. Contrary to the Sambre data, the Lüneburg ship lift data show blocking ratio of 76% (($k = 0.76$) that is higher than for Lock Delden.

Verification and Calibration			Lunenburg	
sailing distance 100 m			trough-in	trough-out
Ship lift	Length	[m]	100	100
	Width	[m]	12.25	12.25
	water depth in lift	[m]	3.41	3.41
Vessel	Beam	[m]	11.40	11.40
	Draft	[m]	2.80	2.80
	Blocking ratio	[-]	0.76	0.76
	Sailing time (over 90 m)	[s]	360	402
	Lunenburg measured speed	[m/s]	0.28	0.25
	Lunenburg; Schijfs method v_{op}	[m/s]	0.31	0.31
	Error w.r.t. measurment	[%]	12%	25%
	Lunenburg; Schijf+Adaption v_{op}	[m/s]	0.27	0.24
	Error w.r.t. measurment	[%]	-4%	-5%

Table 10: Measured and calculated vessel speed for the Lüneburg ship lift

Table 10; Figure 14 and Figure 16 show good agreement between the measured and calculated data, and also that the adaptation of Schijf's method significantly reduces the error w.r.t. the measurements.

6.5 Conclusions on the verification with data from Sambre locks and Lüneburg ship lift

From the verification of the adaption of Schijf's method with data from the Sambre locks and the ship lift at Lüneburg the following conclusions can be drawn:

- The data sets from the Sambre locks cover a blockage range ($0.46 < k < 0.63$) that is not covered by the data set from Lock Delden; and the data sets from the Lüneburg ship lift cover a blockage range ($k > 0.75$) that is also not covered by the data set from Lock Delden;
- The three data sets cover together a wide range of blockage ratios $0.46 < k < 0.76$.
- The three data sets indicate that the vessel speed for blockage ration of $k > 0.6$ can be accurately calculated with the proposed adaption of Schijf's method.
- For smaller blockage ratios ($k < 0.6$) the calculated vessel speed forms an upper boundary for the vessel speed.

7 SUMMARY AND CONCLUSIONS

7.1 Situation

The increase of vessel sizes puts pressure on the waterway authorities to allow larger vessels in the existing infrastructure. The Twentekanaal in the Netherlands is in a process of upgrading from CEMT Class IV to Class Va. Therefore, the clearances in the Lock Delden will in future become below the minimum clearances that are recommended in the guidelines (RVW,2017).

The situation of Lock Delden has been investigated in several steps using desk studies backed up with prototype measurements. The conclusions of each step have been used to evaluate the requirements for the next step.

7.2 Initial desk study

The initial study comprises a desk study on the behaviour of a vessel sailing with very small clearance in the confined water of a navigation lock. Schijf's method has been applied for the calculation of the sailing speed and water level depression around the vessel. The desk study resulted in the following conclusion for the Class Va vessel in the existing Lock Delden:

- The beam clearance and the under-keel clearance become less than recommended;
- The minimum blockage ratio increases to 80% where special measures are recommended for a blockage above 75%.
- The maximum operational vessel speed reduces from 0.65 m/s to 0.25 m/s.
- The maximum lock entry time is more than doubled and with a loaded class Va vessel the lock-Cycle time increases with 11 minutes;
- The limiting speed of and the return flow and the water level depression around the Class Va vessel are significantly less than around the Class IV vessel.

The above results of the desk study indicated that:

- Special measures might be necessary to accommodate Class Va vessels in Lock Delden;
- The decrease of the vessel speed and the subsequent increase of the lock-cycle time might have an impact the capacity of the lock.

The above vessel clearances and vessel speed, together with the uncertainty in the application of Schijf's method for the very confined water in a lock situation lead to the decision to perform prototype measurements in Lock Delden.

7.3 Prototype measurement Lock Delden

- Prototype measurements were performed in Lock Delden with a Class Va vessel with draft of 2.6 m and with draft of 2.8 m, in both sailing directions.
- The skipper used the engine to dead-slow and during the lock entry and departure manoeuvres he did not experience any specific problem;
- The vessel speed computed with Schijf's original method appear to differ up to 28% from the measured vessel speed;
- The vessel speed computed for sailing in the upper head is lower than measured;
- The vessel speed computed for sailing in the lower head is higher than measured;
- The lock entry manoeuvre takes less time than the lock departure manoeuvre.

The above results of the prototype measurements indicate that:

- With the limitation of the use of the engine to dead-slow it appeared to be possible to perform the lock entry and departure safely, even with a blockage of k=0.75;
- The calculations with Schijf's original method resulted in a systematic deviation from the measured vessel speeds. Therefore, Schijf's method should be adapted to counteract these deviations

7.4 Adaptation to Schijf's method

The systematic deviation between the measured and the calculated vessel speed can be explained from hydraulic processes in locks that are not accounted for in Schijf's original method:

- The lock is a dead-end waterway;
- The velocity of the return flow in the narrow clearance between the vessel and the lock is high and the resulting water level slope in the lock becomes significant.

The present paper proposes to adapt Schijf's method on the above two aspects in locks. From the vessel speed computed with the adapted method the following conclusion are drawn:

- The proposed adaption on Schijf's method results in significant smaller differences (5% or less) between the measured and calculates lock entry and departure speed.
- The maximum error has been reduced from 28% to 5% of the measured vessel speed;
- The measurements in Lock Delden cover the blockage ratio: 64% to 75% ($0.64 < k < 0.75$).

The above conclusions are based on eight measurements from Lock Delden.

7.5 Comparison with data from locks in the Sambre river and the Lüneburg ship lift

Measurements in locks in the Sambre comprised ten lock and two ship lift entry or departure manoeuvres. From the calculation of the blockage ratio and vessel speed in the locks the following conclusion can be drawn:

- The blockage ration in the Sambre locks varied between 46% to 63% ($0.46 < k < 0.63$), a range that was not yet covered by the data from Lock Delden;
- For low blockage ratios ($k < 0.6$) the measured vessel speed is lower than computed;
- The manoeuvring actions indicate that the skipper used the main engine in the Sambre locks to dead slow and sparsely. This could explain why the vessel speed is lower than calculated.
- The blockage ration in the Lüneburg ship lift is 76% ($k = 0.76$), just above the range that was not covered by the data from Lock Delden; the adaption on Schijf's method results in similar error as for Lock Delden (5% or less) and a similar reduction compare to Schijf's original method.

7.6 Final conclusion on the proposed adaption of Schijf's method for locks

A simple method has been developed to evaluate the sailing speed and time required for the entering and departing of the lock chamber. The basis of the calculation method is Schijf's well-known method that describes the hydraulics around a vessel sailing at constant speed in a prismatic canal.

For vessel sailing in locks the hydraulics conditions deviate from the assumptions of Schijf's method. Two adaptations have been proposed to account for hydraulic aspects that are not accounted for. The adaptations to Schijf's methods for the calculations of the vessel speed in a lock have been verified for eight measurements in Lock Delden, ten measurements in locks in the Sambre river and two measurements in ship lift Lüneborg. The following final conclusions were drawn:

- The vessel speed in the confined water in locks is significant affected by the dead-end waterway of the lock and the hydraulic resistance (and water level slope) in the return flow. The original version of Schijf's method does not account for these specific hydraulic processes in locks.
- To account for these hydraulic aspects in locks, two adaptations have been proposed to Schijf's original method.
- Calculations with the adaptions show a very good agreement with measurements from situations with high blockage. For the blockage ration of 60% or higher ($k > 0.6$) the error in the calculated vessel speed is less than 5%.
- For blockage ratios of less than 60% ($k < 0.6$), or a calculated vessel speeds of 0.8 m/s (3 km/hr) or higher, the method gives the upper boundary for the vessel speed in the lock.

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CONSTRUCTION OF DAM 1E FOR THE PANAMA CANAL PACIFIC ACCESS CHANNEL

by
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ABSTRACT

To provide navigation access from the new Post-Panamax locks to the original cut section of the Panama Canal, a new 6.7-km-long channel, known as the Pacific Access Channel, was built at the Pacific entrance to the Canal as part of the recent expansion of the historic waterway. The new channel required construction of four dams, referred to as Borinquen Dams 1E, 2E, 1W, and 2W. The dams retain the Canal waters approximately 11 m above the level of Miraflores Lake and 27 m above the Pacific Ocean.

The largest of the dams, Dam 1E, is 2.4 km long and up to 32 m high. The dam abuts against Fabiana Hill at its southern end, and against the original Pedro Miguel Locks at the northern end. This paper provides an overview of the construction of Dam 1E including the sequencing of the works, borrow of the embankment materials from required channel excavations and other sources, and key aspects of construction of the embankment, its foundation, and seepage cutoffs. The paper also describes the most salient design changes required during construction.

1. INTRODUCTION

To provide navigation access from the new Post-Panamax locks to the original cut section of the Panama Canal, a new 6.7-km-long channel, known as the Pacific Access Channel (PAC), was built at the Pacific entrance to the Canal as part of the recent expansion of the historic waterway. The new channel required construction of four dams, referred to as Borinquen Dams 1E, 2E, 1W, and 2W. The dams retain the Canal waters approximately 11 m above the level of Miraflores Lake and 27 m above the Pacific Ocean. A view of the 5-km-long, northern portion of the new channel and of Dam 1E is shown in Figure 1.

The dams were designed as zoned rockfill embankments with central impervious cores of residual and alluvial soils flanked by filter and drain zones of processed sands and gravels (Mejia et al, 2011). Design and construction of the dams posed multiple challenges, including: 1) variable foundation conditions with occasional weak features, 2) availability of core materials limited to primarily residual soils derived from rock weathering, 3) a wet tropical climate with a 4-month-long dry season, and 4) geologic faults across the dam foundations.

The largest of the dams, Dam 1E, was built as part of a fourth contract for construction of the PAC, which included excavation of 3.8 km of the new channel. The contract was awarded in January 2010 to a consortium of ICA of Mexico, FCC of Spain, and MECO of Costa Rica (CIFM). URS (now AECOM) was engaged by the Panama Canal Authority (Autoridad del Canal de Panamá, ACP) to assist ACP with inspection and design engineering services during construction of Dam 1E.

This paper provides an overview of the construction of Dam 1E including the sequencing of the works, borrow of the embankment materials from the required channel excavations and other sources, and the key aspects of construction of the above project elements. The paper will also describe the most important design changes required during construction of the dam. A full description of the dam construction is provided by URS (2015a).

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Figure 1. View of Pacific Access Channel (PAC) and Dam 1E, looking North

Construction of Dam 1E included the following main project components: 1) a 1.7-km-long, 19-m-high, cellular sheetpile cofferdam, 2) a 30-m-long, 18-m-deep, triple-row, jet-grout cutoff wall, 3) a 460-m-long, 18-m-deep, cement-bentonite slurry cutoff wall, 4) dewatering and excavation of the dam foundation, 5) treatment and geologic mapping of the foundation, 6) a 2.4-km-long, double-row grout curtain, 7) a 2.4-km-long, 5.3-million-cubic-meter, zoned rockfill embankment, 8) performance monitoring instrumentation, and 9) a 97-m-long, 26-m-deep, secant-pile wall to provide closure against the structure of the Pedro Miguel Locks.

2. DAM DESIGN

Borinquen Dam 1E was designed as a central earth core and rockfill embankment. To adequately control potential seepage and internal erosion of the core and to provide internal drainage, zones of chimney filters and drains were designed to flank the core. In addition, filter and drain blankets extend along the entire foundation of the outboard (downstream) rockfill shell to control internal erosion of the foundation. A full description of the design concept and rationale is provided by Mejia et al. (2011).

The dam is 2,420 m long and has a crest 30 m wide at elevation 32.30 m (project datum is close to mean sea level, MSL). The southern end of the dam abuts against Fabiana Hill, and the north end is connected to the northern monolith of the Pedro Miguel Locks by a 97-m-long secant pile cutoff wall, known as the North Tie-In wall.

Figure 2 shows the general arrangement of Dam 1E and the PAC. A typical cross section of the dam is shown in Figure 3. The as-built embankment volume is approximately 5,290,000 m^3.

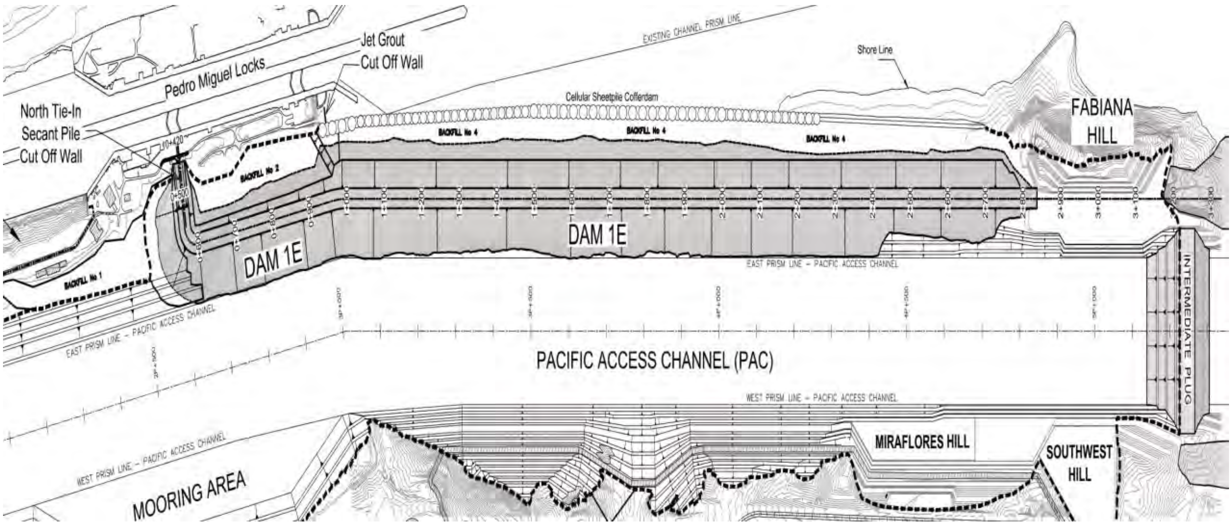

Figure 2. Layout of Borinquen Dam 1E and Pacific Access Channel (PAC)

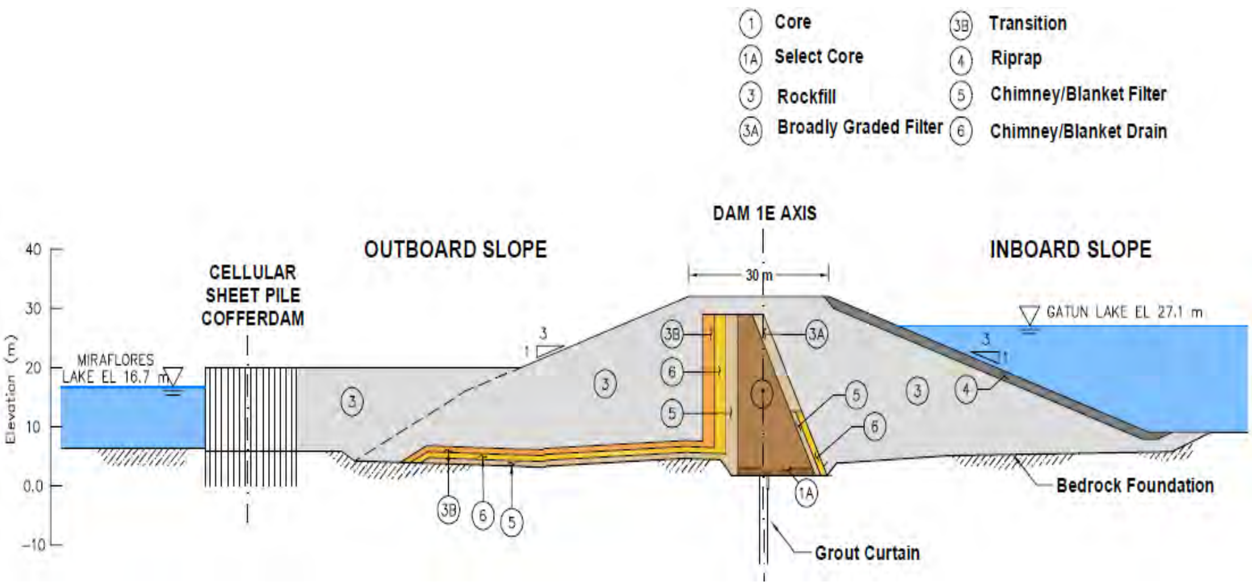

Figure 3. Typical Cross Section of Borinquen Dam 1E

3. CELLULAR COFFERDAM AND CUTOFF WALLS

A cellular sheet-pile cofferdam and a jet-grout wall were constructed to isolate the construction area from Miraflores Lake, and allow dewatering of the PAC and dam foundation excavations. In addition, a cement-bentonite cutoff wall was installed to reinforce the existing Pedro Miguel Saddle dam, which extends between the structure of the Pedro Miguel Locks and the foot of the former Paraiso Hill.

3.1 Cellular Sheet-Pile Cofferdam

The cofferdam extends along the outboard toe of Dam 1E from the southern wingwall of the Pedro Miguel Locks to Fabiana Hill (Figures 2 and 3). It consists of a 1,300-m-long section of sheet-pile cells and a 460-m-long section of interlocked Z-type sheets. The latter section provides a cut-off through fill that extends to the foot of Fabiana Hill. The cells have diameters between 17.9 m and 21.8 m, and a height of 19 m. Circular cells are connected by intermediate arc cells of compatible diameters.

Construction of the cofferdam began in August 2010 with dredging of the foundation to remove soft lake-bottom sediments over a 1,100-m length of the structure. The sheet-piles were aligned using a steel template, and were inserted progressively by self-weight, by driving with a vibratory hammer, and by a diesel impact hammer. The sheets were driven to their design tip elevation of -1.0 m or to refusal in rock. Unsuitable materials were removed from within the cells, and these were backfilled with free-draining gravel (Figure 4). After backfilling, the west face of the cofferdam was buttressed with a rock berm placed through water, which was completed in May 2011.

Figure 4. Backfilling of cellular sheet-pile segment of cofferdam

3.2 Jet-Grout Cutoff Wall

At the north end of the cofferdam (Figure 2), a jet-grout wall was installed between the cofferdam and the southern wing-wall of the Pedro Miguel Locks, to provide a cutoff to seepage from Lake Miraflores. The cutoff consisted of a 30-m-long, 18-m-deep, triple-row, jet-grout wall. The inboard (upstream) row was constructed first, followed by the outboard row, in turn followed by the middle row. Three passes were made for each row to install primary columns followed by secondary columns. Grout was injected at a rate of 350 kg/m (controlled through the rotation and rise rate of the drill rod) at a pressure of approximately 350 bars.

3.3 Cement-Bentonite Cutoff Wall

The Pedro Miguel Dam closed the former gap between Paraiso Hill and the Pedro Miguel Locks to retain Gatun Lake. Because this dam was not designed for the water head expected during construction, a cutoff wall was required to control seepage and prevent piping through the dam and its soil foundation, and thus, prevent loss of Gatun Lake. This cutoff is approximately 458 m long and 18 m deep.

The wall was excavated and constructed in panels using a clamshell. A cement-bentonite slurry was continuously pumped into each panel during excavation until the final depth was reached. The wall penetrated through the dam fill and underlying alluvial soils to terminate at least 1 m into La Boca Formation rock.

4. CHANNEL AND DAM FOUNDATION EXCAVATIONS

Excavation of the PAC began in June 2010 and blasting of rock started the following August. Following completion of the cellular cofferdam, the dam footprint was dewatered. Removal of muck from the former lake bed commenced shortly afterwards. Bulk excavation of the dam foundation started near the south end and progressed north. The foundation excavation was nearly complete by late 2012. Initial excavation was stopped short 1 to 2 m above the foundation objective level. The materials above this level were removed during final excavation. Figure 5 shows the dam foundation after bulk excavation and initial placement of rockfill on the inboard foundation. The volume of PAC and dam foundation excavation totaled 26,989,000 m^3.

Figure 5. Dam 1E foundation with cellular cofferdam (left), PAC excavation (right), and rockfill shell on the inboard dam foundation

5. DAM EMBANKMENT MATERIALS

The dam embankment materials were sourced from the channel excavation and reserve borrow areas of residual soil. Additional volumes of residual soils, rockfill, and sound basalt for aggregate production were obtained from external sources. An additional 3,910,000 m^3 was obtained from sources outside the channel limits, as modifications to the contract.

5.1 Rockfill

Rockfill was obtained from the channel excavations, Miraflores Hill (Figure 2), and disposal stockpiles from previous PAC excavation contracts. The Basalt and Pedro Miguel Formations encountered in the channel yielded suitable rockfill (Zone 3 in Figure 3). The basalt was a medium hard to very hard, strong to very strong, massive or columnar jointed rock. The welded tuff and agglomerate facies of the Pedro Miguel Formation were also strong and sufficiently durable for use as rockfill. The fine-grained tuffaceous facies of that formation were found to be weak and subject to slaking under repeated cycles of wetting and drying, and were excluded from use in dam construction.

Approximately, 1,571,000 m^3 of rockfill was obtained from disposal stockpiles left by earlier PAC excavation contracts. Those stockpiles contained a mix of basalt and welded agglomerate rock. The disposal stockpiles were also used for backfill between the dam embankment and the cofferdam (Figure 3).

5.2 Core

The core material (Zone 1 in figure 3) was sourced from alluvial and residual soils derived from the weathering of local bedrock. The alluvial sources consisted of deposits excavated from the dam foundation and from a reserve borrow area adjacent to the Cocoli River. The alluvial soils classified as clays, sandy clays, silts, and sandy silts of high plasticity (CH/MH). The natural water content of the alluvial soils was generally above the standard optimum, up to about 12%.

Residual soils of Pedro Miguel agglomerate and of basalt were sourced from the channel excavations and from the Miraflores Hill 2 borrow site, respectively. Residual soils from siltstones and sandstones of the La Boca Formation and from diorite were obtained from other borrow sites. A source of La Boca Formation residual soil several meters thick was developed on the east side of the Canal, approximately 5.0 km north of the site. The residual soils often classified as a sandy silt of high plasticity (MH), plotting just below the A line on the Casagrande plasticity chart.

The core soils were mined with excavators, hauled to stockpiles in 40-ton articulated dump trucks, and spread with bulldozers in 300-mm lifts. The alluvial borrow sources provided relatively homogeneous materials that required little mixing. The residual soil borrow sources were more variable and required mixing and removal of oversize rock. As the stockpiles were raised, they were sealed with a smooth drum roller, and slope-graded to minimize rainwater infiltration. The alluvial soils were typically at the upper limit of the water content specification and workability. The in-situ water content of the residual soils was often drier than the minimum specified, requiring addition of water in the stockpile.

Ten test fills were completed on the core materials, one for each borrow source, to demonstrate compliance with the specifications. They consisted of a minimum of three lifts for compaction and testing. The water content of the lifts was progressively increased to find the upper limit at which the minimum specified undrained shear strength of the compacted material could be achieved.

5.3 Filter and Drain

Filter and drain materials were produced by onsite crushing and screening of basalt excavated from the channel and were also imported from offsite sources. Up to seven crushing plants were mobilized for the on-site production of fine filter (Zone 5 in figure 3), drain (Zone 6 in figure 3), and transition (Zone 3A/B in figure 3) materials. To meet the embankment demand, Zone 5 and Zone 6 materials were also obtained from offsite quarries.

Considerable difficulties were encountered in configuring and calibrating the onsite plants, and in selecting suitable feed materials to achieve specification compliance. Compliant production of Zone 5 materials began in January 2012, nearly a year after equipment mobilization. To facilitate production, the Contractor requested a few changes in the specified gradations, basically increasing the maximum fines content from 3 to 5% in Zone 5, from 2 to 3% in Zone 6, and from 3 to 5% in Zone 3A.

To verify compliance with the specification, quality control testing was performed at the plant, at the stockpile, and within the placed lifts in the embankment. Final approval of material was based on gradations taken from material placed in the embankment.

6. FOUNDATION WORKS

Dewatering of the foundation excavation was accomplished by pumping from 150-mm-diameter deep wells with tip elevations of -10 m. The wells were successful in drawing down the groundwater to at least 1 m below the base of the core and allowing preparation of the foundation. Groundwater levels were monitored with standpipe piezometers distributed across the foundation. All wells were decommissioned by removal of the screens and tremie or pressure grouting of the holes.

Excavation within the core foundation commenced in May 2012 and continued until September 2014. During excavation of the core trench, several sections of the outboard cut slope slumped. These failures required regrading the slope locally to inclinations between 2H:1V and 3H:1V to remove the failed masses and stabilize the slope. No failures occurred on the inboard slope. The slumps developed on adversely oriented features such as bedding shears or joints, which caused wedge failures of the trench walls (Figure 6).

Figure 6. Example failure of outboard core trench slope

6.1 Geologic Mapping

Dam 1E is underlain predominantly by Miocene-age sedimentary rock of the La Boca Formation and volcanic rocks of the Pedro Miguel and Basalt Formations. The sedimentary rock is of low-strength and consists of interbedded, sandstone, siltstone, clay shale and conglomerate units, and underlies the northern three-quarters of the dam foundation. Near the southern end, the La Boca Formation is overlain by interbedded volcaniclastic sandstone, with minor siltstone units, of the Pedro Miguel Formation (URS, 2015b).

The foundation geology was mapped to confirm compliance with the specification and to determine treatment requirements (Figure 7). After treatment and final cleaning, the foundation surface was inspected and approved for placement of embankment fill. Where geologic units at foundation grade did not meet the specified requirements (e.g. some weak lignite beds and sheared clay shale beds) foundation treatment was specified.

Figure 7. Geologic mapping of the dam core foundation

6.2 Design Changes

6.2.1 Core trench widening at fault crossings - Based on the geologic investigations carried out for design, two branches (East and West) of the Pedro Miguel Fault capable of such displacement were mapped to cross the dam foundation during design (Mejia et al., 2011). The design of Dam 1E included provisions to widen the chimney filter and drain zones within 50 m of fault traces deemed capable of the design fault displacement.

Geologic mapping during construction revealed multiple minor faults throughout the foundation (URS, 2015b). A prominent fault trace was encountered crossing the dam axis about 200 m north of where the West Branch of the Pedro Miguel Fault was mapped before construction (Schug et al., 2016). Two fault traces associated with the Pedro Miguel fault were encountered near where the East Branch of the fault had been mapped at the foot of Fabiana Hill. Those faults were judged capable of the design fault displacement and the filter and drain widening provisions of the design were implemented at the exposed crossings of the faults, resulting in widening of the core trench at those locations.

6.2.2 PAC cut slope instability - Immediately after initial excavation, instability developed over a 100-m length of the PAC cut slope along the inboard toe of Dam 1E, about one-third of the dam length north of Fabiana Hill (Figure 2). The cut slope failed towards the floor of the channel leaving back-scarps extending approximately 5 m back from the top of the cut, close to the design location of the dam toe.

Subsequent field investigations showed that the slope was cut in weak, fissured rock with adverse dips associated with faulting. Investigation trenches indicated that the failures were translational slumps with movement on basal bedding planes out of slope. These geologic conditions were deemed to pose a significant instability hazard to the dam. Thus, the foundation excavation design was revised over a 550-m length by removing the sheared rock and excavating a wide bench 2 m below the level of the PAC floor along the inboard dam toe. The excavation was backfilled with compacted rockfill, which became part of the inboard dam shoulder. The required bench width varied between 20 and 35 m, based on stability analyses using selectively fully-softened and residual strengths for various foundation rock features.

7. FOUNDATION GROUTING

The grout curtain extended the full length of the dam to a nominal depth of 15 m. The curtain consists of two rows of holes. One row runs 1.5 m inboard (Row B) of the dam axis and the other 1.5 m outboard (Row A). The Row-A holes dip 70 degrees to the north whereas the Row-B holes dip to the south. A 3-m-deep concrete cutoff wall was installed along Row B. In addition, multiple rows of “stitch grout” holes were installed in shear zones across the dam axis (URS, 2015c).

Super primary holes were drilled at 24-m intervals along the curtain, to a 25-m depth below the foundation. Primary holes, spaced at 6 m on centers, and secondary holes (at the mid points) were mandatory, resulting in a maximum 3-m spacing between holes along both rows. The design provided for “split-spaced” holes to be triggered if the grout take in any of the mandatory holes exceeded a predetermined quantity.

The grout was specified to consist of Type III cement, API 13A grade bentonite, potable water, superplasticizer, and silica fume. Holes were grouted using an ascending stage split-space method. Grouting pressures of 28.3 kPa/m were used for depths less than 18 m, and of 45.2 kPa/m, for greater depths. As foundation conditions became better understood and survey data became available, grouting pressures were adjusted to manage ground movement. Reduced grouting pressures were used at the south abutment.

A closure criterion of 25 kg of cement injected per meter of grout stage was established initially for grout take. Based on verification testing, which indicated no discernible decrease in the effectiveness of the curtain, that criterion was revised to 35 kg/m, to improve production. Criteria such as communication between holes, surface leakage, and hole collapse were used to add split-spaced holes.

Grouting work started in late May 2012 with a 30-m-long test section. The curtain was completed in December 2014. Approximately 5950 holes were drilled and grouted, including stitch grouting, and 2,842,000 kg of cement were injected into the ground.

8. FOUNDATION TREATMENT AND CLEANING

Treatment of the core foundation was applied in two stages. To prepare fault and shear zones for stitch grouting, initial rock shaping and dental concrete were completed before grout curtain installation. The final treatment, performed after the grout curtain was completed, consisted of minor dental concrete, shotcrete, and slush grouting.

Cleaning of the foundation surface to remove loose and encrusted materials was achieved with compressed air and water. The fine grained, non-welded clay shale units within the La Boca formation deteriorated rapidly after exposure. This required trimming and cleaning of the foundation immediately before placement of embankment materials.

8.1 Concrete Cutoff Wall

Following initial cleaning and mapping of the core foundation, the concrete cutoff wall was installed along Row B of the grout curtain. The wall is 3 m deep and was extended to the full length of the foundation.

The wall trench was excavated with a trenching machine with a 0.65-m-wide blade. Compressed air, water, and suction were used to remove rock fragments, water, and debris at the base of the trench prior to backfilling with concrete (Figure 8). Adversely oriented discontinuities (bedding, joints, faults) in the bedrock caused multiple blocks and wedges up to 2 m wide to collapse from the outboard wall at the top of the trench. These collapses were typically repaired with dental concrete.

Figure 8. Cleaning of concrete cutoff wall trench

8.2 Shear Zones

The core foundation contained numerous shear zones and faults ranging from less-than-50-mm-wide shear zones to sheared rock and gouge zones up to 1.5 m wide. The gouge materials typically consisted of low plasticity silt and sandy soil, and included up to 5-cm-thick seams of high plasticity clays and silts, in which polished surfaces could be separated by hand. The shear zones and faults were treated by over-excavating them to a depth three times their width, up to a maximum depth of 1.5 m. The features were cleaned with compressed air or water and backfilled with concrete.

8.3 Weak Beds

Beds of highly fractured, weak rock were encountered in the La Boca Formation, generally in the finer grained units (clay shales). The beds were often sheared through their full thickness with polished surfaces. Several beds of dark gray to black, weak, carbonaceous clay shale and siltstone, interbedded with lignite, were also encountered (Figure 9). Generally, the rock quality of these beds did not improve with depth. To provide a suitable surface for placing compacted core, these weak beds were covered with a minimum thickness of 0.2 m of backfill concrete. Shotcrete was used on inclined fractured and weak rock surfaces, where regular concrete could not be vibrated in place.

8.4 Shoulder Foundation Treatment and Cleaning

Once suitable rock was exposed, the outboard shell foundation was cleaned by trimming with an excavator fitted with a smooth-edged bucket. Significant shear zones and faults were treated by over-excavating the sheared materials to a depth three times the width of the feature and backfilling the excavation with compacted filter material. The excavation depth was generally limited to 1.5 m for large features.

Figure 9. Cleaning of lignite bed for placement of backfill concrete

9. EMBANKMENT CONSTRUCTION

9.1 Construction Sequence

Placement of the core was sequenced to start after the nearby grout curtain was completed. Therefore, the inboard shell was placed up to a height of 17 m in advance of the core, over a significant length of the dam. Once grouting was completed, core placement commenced in July 2013 at the northern end of the dam, and progressed south. A second core placement front was started in December 2013 near the south end (Figure 10). The outboard filter blanket was started in August 2013 in two simultaneous fronts, north and south. Embankment construction was completed in July 2015.

The chimney filter zones were generally placed ahead of the adjacent core. After placing a lift of filter material, a lift of core was placed in between the filter zones. Upon placement of the core lift, the adjacent rockfill zones were placed.

Figure 10. Dam embankment construction, looking north from Fabiana Hill

9.2 Embankment Core

A zone of select core material (Zone 1A) up to 1 m thick was placed at the base of the core to provide a good contact with the foundation. Alluvial soil without rock fragments was typically used for this zone. A 300-mm-thick lift was placed first to avoid damaging the foundation. A 200-mm loose thickness was used for subsequent lifts within Zone 1A. The materials were compacted with 8 passes of a weighted front-end loader (Figure 11). In confined areas inaccessible to the loader, gasoline-powered hand tampers were used with a reduced lift thickness of 150 mm.

The main body of the core (Zone 1) was placed in 225-mm-thick loose lifts. The earthfill was hauled to the embankment from the stockpiles in 40-ton articulated dump trucks, and was dumped and leveled to the required lift thickness by dozers. Each lift was disked and compacted with a self-propelled, quadruple-drum, pad-foot compactor or with a rubber-tired, single-drum, pad-foot roller.

The specified upper limit on compaction water content was +12% above the standard optimum. However, the earthfill became effectively unworkable at a water content of approximately +8%, and required removal of lifts exhibiting deep rutting (close to or deeper than the thickness of the lift). Furthermore, at water contents higher than +8%, the minimum specified undrained shear strength of 75 kPa could not be achieved reliably.

Figure 11. Compaction of core Zone 1A with front-end loader

A substantial portion of Zone 1 had to be placed during the wet season, making control of water content difficult. Compaction and sealing of lifts with a cross fall, prior to approaching rains, proved effective in limiting ponding of water and infiltration of rainfall on the core working surface. Despite these measures, more than 40,000 m^3 of core material had to be removed from the fill and returned to the stockpile for re-conditioning.

Compaction control was achieved by field testing and inspection. Compaction was indexed by measuring the undrained shear strength of each lift using a field shear vane in accordance with standard ASTM D2573 (Figure 12). This proved to be a rapid and reliable quality control test method. As part of the quality assurance program, field density tests were also performed using the sand replacement method (ASTM D1556) at the same locations tested with the shear vane.

Figure 12. Vane shear testing of compacted core material

The in-place average water content of Zone 1 was +5.7% above the standard optimum. The average vane shear strength was 114 kPa. The relative compaction of the material averaged 93.2% (ranging between 85 and 99%) of the standard maximum dry density (ASTM D698).

9.3 Filter and Drain Zones

In accordance with the design concept (Mejia et al., 2011), the chimney filter, drain, and transition zones flanking the core were widened from their design width of 2.5 m to 4.5 m within 50 m of the main fault crossings. Depending on access constraints, the materials were placed by one of the following methods: a) spreading with an excavator, b) end-dumping from haul trucks, and c) delivery by telescoping belt and tremmie hose (Figure 13). Once spread, the materials were leveled in 300-mm-thick lifts and compacted by 4 passes of a 12-ton, smooth-drum, vibratory, self-propelled roller. Ample water was added to the materials during placement and spreading.

Protective mats were placed to allow haul trucks to cross over the chimney filters and drains. These methods proved effective at minimizing contamination and breakdown of the materials. Traffic over the blankets was avoided to the extent possible.

Figure 13. Placing filter material with telescoping belt and tremmie hose

9.4 Rockfill

Shoulder rockfill was placed by end-dumping from large trucks on an advancing front and by leveling the material in 0.9-m lifts with a bulldozer. Water was added with hoses or water trucks during spreading (Figure 14). The rockfill was compacted by 6 passes of a 12-ton, smooth-drum, vibratory, self-propelled roller. Eleven in-situ density and gradation tests were completed in the rockfill. An average dry density of 2,310 kg/m^3 was measured using standard ASTM D5030.

Figure 14. End-dumping, spreading, and watering of shoulder rockfill

9.5 Riprap

Riprap (Zone 4 in figure 3) was placed to protect the inboard face of the embankment from erosion due to wave loading and water drag forces induced by vessel transit. Riprap was sourced by selectively stockpiling columnar basalt excavated from the PAC, Miraflores Hill, and Southwest Hill (Figure 2). The material was placed using a 30-ton excavator (Figure 15) after trimming the rockfill slope to allow accurate placement of the riprap.

Figure 15. Riprap placement on trimmed face of rockfill

10. NORTH TIE-IN CUTOFF WALL

The North Tie-In Cutoff Wall completes the Dam 1E water barrier between the dam embankment and the Pedro Miguel Locks. The wall is a plastic concrete secant-pile cutoff, 97 m long and up to 26 m deep. It penetrates at least 1 m into sound rock of the La Boca Formation, and overlaps vertically with the grout curtain to cut off seepage through the backfill of the locks structure and the underlying bedrock.

The design required that the secant-pile layout provide a minimum wall thickness of 0.6 m. The Contractor selected a pile diameter of 1.4 m with a center-to-center spacing of 0.86 m, resulting in an overlap thickness of 1.1 m. This layout provided ample margin to meet the specified minimum wall thickness, while allowing for potential deviation of the piles from vertical.

Plastic concrete with an unconfined compressive strength between 750 and 1400 kPa, and a maximum permeability of $5x10^{-7}$ cm/sec at 28 days, was specified as the wall material. The Contractor carried out a laboratory test program and selected mix a consisting of 437 kg of water, 29 kg of bentonite, 140 kg of cement, and 1340 kg of aggregate per cubic meter. This mix yielded a 28-day compressive strength between 1080 and 1100 kPa, and a permeability between $3.6x10^{-7}$ and $5.1x10^{-7}$ cm/sec.

A drill rig equipped with a cutter-tooth auger bit and a hydraulic oscillator was used to drill and case the pile holes. The oscillator successfully drove temporary steel casing to support the holes in the cobble and boulder backfill of the Pedro Miguel Locks and the auger excavated the materials out (Figure 16). The shaft bottom was carefully cleaned with a trap-door bucket auger.

Figure 16. Excavation of secant piles

A mobile concrete plant was used to batch the plastic concrete and dispatch the materials into concrete agitator trucks that finished mixing the batch components before tremie-placement in the piles. During pile production, the batch plant was unable to maintain calibration, creating variability in the strength and permeability of the concrete. Fourteen piles out of 112 were replaced for strength, permeability or other deficiencies.

Closure of the secant pile wall against the concrete monolith of the locks was made by two closure shafts drilled against the monolith. One of the shafts was re-drilled to provide a broad contact with the monolith (Figure 17). The gap between the closure shafts and the monolith wall was closed with 850-mm-OD connection shafts. The connection shafts were cored to intersect each other and to overlap the monolith wall and the closure shafts. Camera inspection showed that the north closure shaft deviated from the monolith wall, and an additional connection shaft was drilled to ensure a complete seal (Figure 17). The closure and connection shafts were backfilled with lean concrete.

Upon completion of the wall, the top of the wall was trimmed to the design elevation and the embankment was completed over the wall alignment. The joints between piles were delineated at the surface by a thin smear of clay/bentonite around the perimeter of the secondary piles (Figure 18). The contact between piles and the integrity of the piles was verified at depth with drillholes. Core recovery and camera inspection of the holes showed a good contact between piles, and between the foundation rock and the base of the piles.

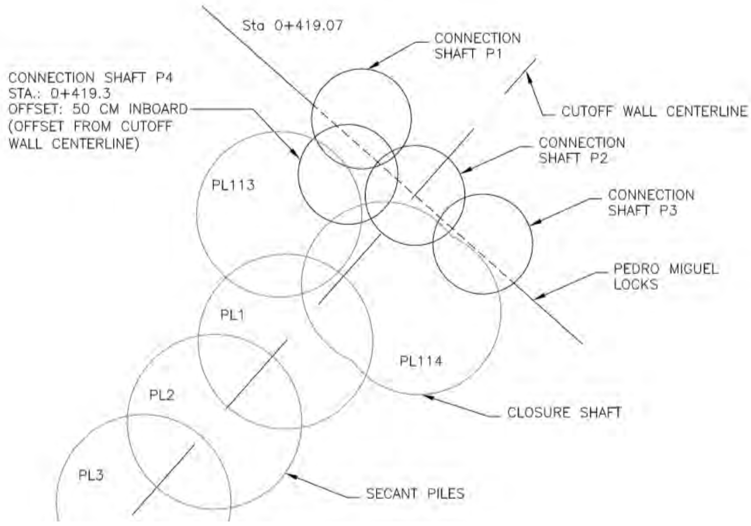

Figure 17. Layout of closure and connection shafts of secant-pile wall

11. INSTRUMENTATION

The design called for installing instrumentation to monitor the performance of the dam: a) as the embankment was completed, b) during filling of the PAC, and c) during long-term operation of the dam. The following instrumentation was installed as embankment construction progressed:

- Surface survey monuments at 50 m intervals along the inboard crest edge and along the embankment downstream toe, to measure vertical and horizontal displacements.
- Inclinometers through the dam shoulders at three locations where significant bedding plane shears were observed in the foundation.
- Settlement sensors at two locations along the dam core, to measure settlement at three levels within the core.
- Vibrating wire piezometers at regular intervals along the dam, to measure water pressures in the foundation beneath the outboard shell and dam toe.
- Standpipe piezometers within the fill at the outboard toe, to measure water levels within the outboard backfill.
- Standpipe piezometers to measure groundwater levels within Fabiana hill near the location of the Pedro Miguel Fault, which was treated with stitch grouting.
- Accelerographs on the dam crest, at the Pedro Miguel Locks monolith, and at the Fabiana hill abutment, to record future earthquake accelerations.

Figure 18. Trimming the top of the cutoff secant-pile wall

The settlement sensors and vibrating wire piezometers were routed to terminals on the embankment crest. An Automated Data Acquisition System (ADAS) was installed to allow automated reading of the instruments and wireless data transfer to ACP's offices.

12. CHANNEL FILLING

The north tie-in secant pile wall was the final element of the dam to be completed. After the last piles in the wall were built and allowed to cure sufficiently, filling of the PAC commenced. Filling lasted 101 days including a holding period of 28 days with the water level at elevation 22.0 m. The reservoir water level reached the design elevation of 27.10 m on 16 November 2015, without incident. During the filling period, the instruments were read and the data were reviewed daily. The measured response of the dam to reservoir filling was well within design expectations.

13. CONCLUSIONS

Borinquen Dam 1E is a critical component of the Panama Canal and is vital to the Canal's operation. This paper presented an overview of the construction of the dam, including the sequencing of the works, the borrow of embankment materials, and key aspects of construction of the embankment, its foundation, and seepage cutoffs. Salient design changes required during construction were also described.

Construction of the dam was a complex undertaking involving multiple and diverse project elements. Several challenges were encountered during the work, including: weak foundation conditions and cross-cutting geologic faults, core materials of variable residual and alluvial soils, and a wet tropical climate with a short dry season. These challenges were met and the dam was constructed successfully through: a) close collaboration between the Contractor (CIFM), the Owner and Construction Manager (ACP), and the Design Engineer (URS), b) diligent inspection and quality assurance testing of the works, c) flexibility in the design concept, d) diligent investigation and solution of unanticipated issues, and e) timely decision-making.

The reservoir was filled without incident and the dam has been performing satisfactorily since it was completed in September 2015. The response of the dam to reservoir filling and its performance under the permanent embankment and reservoir loads is consistent with the design assumptions and has been well within expectations.

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WG197 SMALL HYDROPOWER IN WATERWAYS ENVIRONMENTAL WG197

Introduction

Whenever the infrastructure around rivers or canals is modified, improved or even just maintained, additional environmental constraints are likely to be placed on the owners and operators. The development of a new hydro power plant in conjunction with existing infrastructure will undoubtedly raise such issues. Even if the infrastructure has been in place for many years and the introduction of a hydro plant will do little more that divert some of the water in the waterway through a different route around an existing weir, the environmental authorities are likely to insist that the new installation improves the situation for migratory fish rather than just maintaining the status quo.

This paper looks at some of the environmental issues that can arise during the development of hydro power plant on inland navigations and how they might be approached and mitigated.

Environmental legislation and regulation

Every country has its own environmental regulations. Even the countries of the European Union, whilst bound by common EU directives, implement these directives in their own way. There can be regional interpretations of any one country's own regulations so what applies in the south doesn't necessarily apply in the north.

As a result, this paper cannot present solutions that are applicable in every case, but rather a series of solutions that have been used successfully in some countries to give owners and operators a view of the scale of the issues and they ways in which they can be successfully addressed. Every development is different, the actual solution applied in each case will have to take account of the country rules and regulations.

The constrains and requirements of environmental regulations will increase the cost of the hydro power plant development. Sometimes, the increases can start to affect the economic viability of the scheme to the point where its viability is reduced or even eliminated.

The environmental effects can be broken down into a number of areas:

- Fish and eel migration upstream and downstream
- Floating Debris
- Sediment transport and redistribution

Fish and eel migration

Migratory fish and eel species are of considerable importance to the wider environment and, in some cases, are economically significant as well. The fishing of salmon is particularly important in some areas of the Northern hemisphere and the downstream migration of the juvenile fish – known as smolts – is key to the continued survival of the species. Eels have suffered major population declines in recent years and there are legal protections in place to protect eels and ensure that their migrations up and down rivers can take place unhindered.

Upstream migration - fish

The key elements for upstream migration are:

- Provision of passes for fish and eels
- Exclusion of fish and eels from the turbine discharge

The type of fish pass required varies by country and by fish species. There are three basic types of fish passes:

- Pool
- Baffle
- Active

Pool fish passes

A pool fish pass consists of a series of linked pools with water cascading between each pool either over horizontal barriers, through vertical slots or through submerged orifices. The pools separated by horizontal barriers are suitable for fish that can jump or leap over barriers in their progress upstream. The vertical slot variant can have additional materials on the bed to facilitate the movement of benthic (bottom dwelling) fish up the fish pass. In both cases the water velocities, turbulence and pool size are carefully calculated to enable the required fish species to pass through them. There are a number of variants and combinations of vertical slots, submerged orifices and horizontal barriers to make the fish passes applicable to particular fish species.

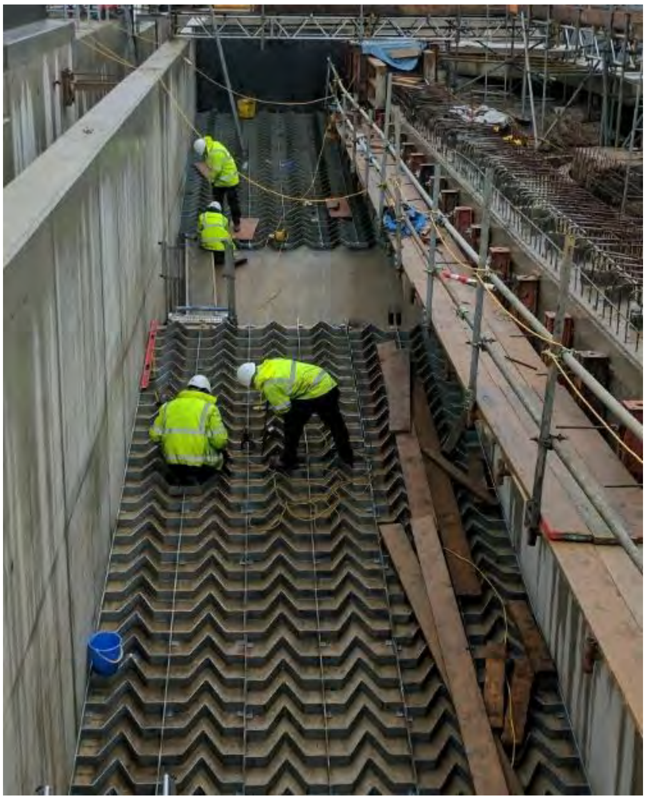
Vee shaped slots, horizontal barriers with notches and submerged orifices that have been used in different counties and the choice is very much country specific. The construction costs of all of these can be a significant proportion of the overall cost of the hydro power station development.

Pool fish pass in France with two vertical slots in each pool. The bed of the fish pass has additional moulded protrusions to assist the passage of benthic fish.

Pool fish pass in Scotland with submerged orifice

Baffle fish passes

Baffle fish passes use a series of shaped steel plates in a concrete trough. Long runs have resting pools part way up. The baffles come in many forms and shapes that are designed to facilitate the passage of various species of fish. In recent years, the Larinier type of super-active baffles have found favour in Europe because of their ability to facilitate the passage of a large number of species of fish.



Larinier super-active baffle type fish pass under construction and in operation on the Aire & Calder Navigation in England – Pictures from Yorkshire Hydropower Ltd.

Active fish lifts

Active fish lifts are mechanised structures that attract fish and then lift them either in the manner of a navigation lock or in a tank that is moved on rails between the lower and upper water levels. These tend to be associated with a large difference in levels such as at hydro-power dams.

Turbine discharges

Migratory fish are attracted to waterflows in their quest to swim upstream. Where the discharge from a hydro power station forms a large part of the flow in the waterway, there is always the possibility of the fish following the turbine flow rather than being attracted to the fish pass flow. It may be necessary to put screens on the turbine outlet to prevent fish trying to follow this flow. Such outlet screens don't need to be raked because the debris is on the downstream side, but they do need to be carefully designed to prevent head loss and the back-up of the water level downstream of the turbine.

Upstream migration - eels

The facilitation of eel migration is of significant concern in Europe. Migrating eels are poor swimmers and cannot use the more traditional types of fish pass. Eel specific passes will be required where eel migration occurs.

Eel passes require a wet substrate that the juvenile eels, or elvers, can wriggle through to climb. Bristle type materials, artificial grass or specifically designed rough plastic substrates that are wetted by a continual flow of water have been successfully used. The amount of water needed is quite small, a few litres per second can provide enough to allow the eels to use the pass.

The pass can either be an open channel or a closed conduit such as a plastic pipe. Careful design is needed to provide an entry and exit from the eel pass to ensure that it is kept clear of debris and doesn't let the emerging eels get swept back over the weir.

Various forms of eel pass

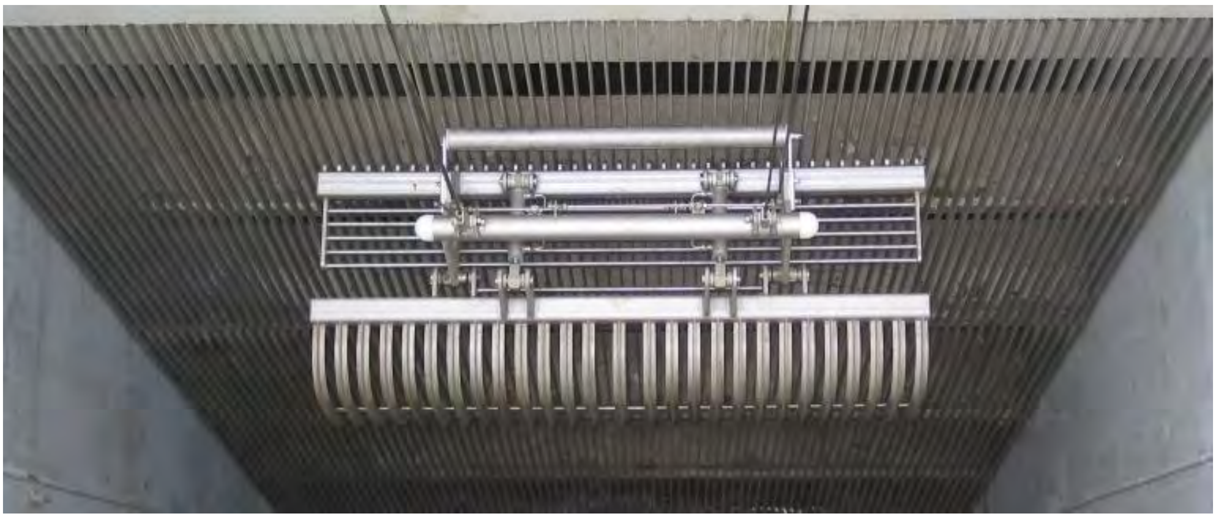
Downstream migration - fish

There are two approaches to dealing with migratory fish going downstream associated with a hydro power plant:

- Exclusion of fish – keeping the fish out of the turbine
- Fish friendly turbine – a significant number of the fish can pass through the turbine unharmed

Exclusion of fish

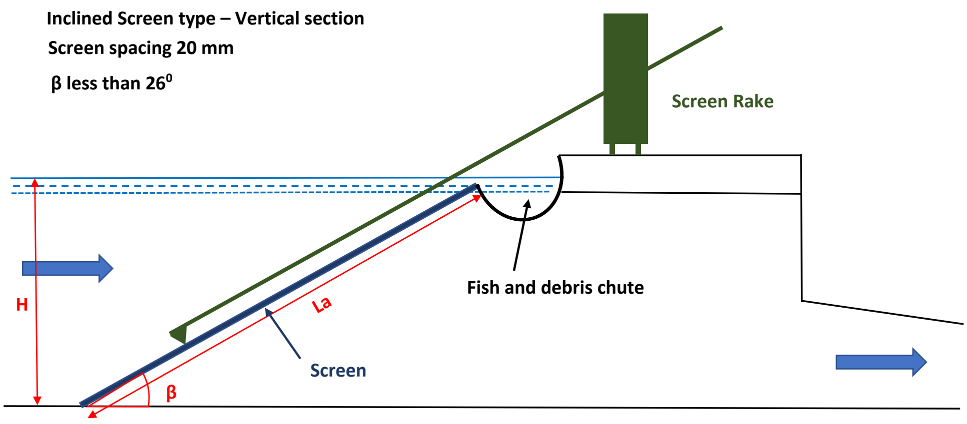
The inlets to all turbines must be guarded or screened to prevent floating debris from entering and damaging the turbine. Traditionally, these screens have been made from sloping panels of steel bars. The gap between the bars is in the range 50mm to 100mm to keep out debris that might cause damage or blocking in the turbine. Such screens can be raked by an automatic screen rake that rakes up the bars lifting the debris up the sloping screen panel to remove debris that could block the screens and increase the head loss on entry to the turbine. The debris can either be lifted out of the water for dry disposal or directed into a channel that sluices it downstream.

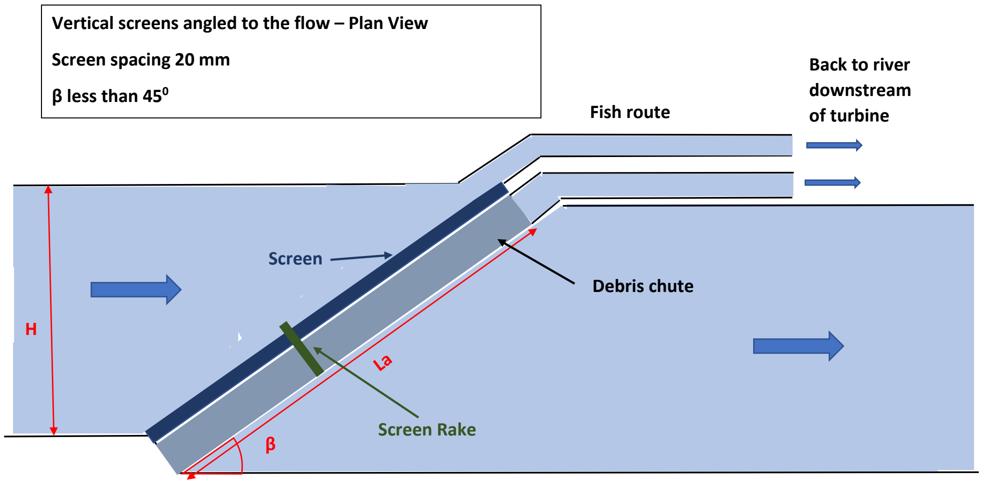


Traditional turbine intake screen with automatic rake

Such screens with wide bar spacing does little to keep fish out especially small juvenile fish such as salmon smolts. To exclude salmon smolts, the bar spacings will need to be 12. This is about the minimum gap that can be raked vertically by a traditional raking machine.

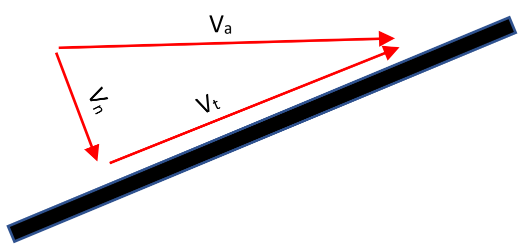
The required gap in such screens is very much the decision of the individual country regulators. In France for example, a minimum gap of 20 mm between the bars is generally used but with specific requirements concerning the angle of the screen to the flowing water, the water velocity normal to the screen and the water velocity on the face of the screen. The screens can be at right angles across the waterway and inclined to vertical as below, or the screens can be vertical and angled across the waterway.



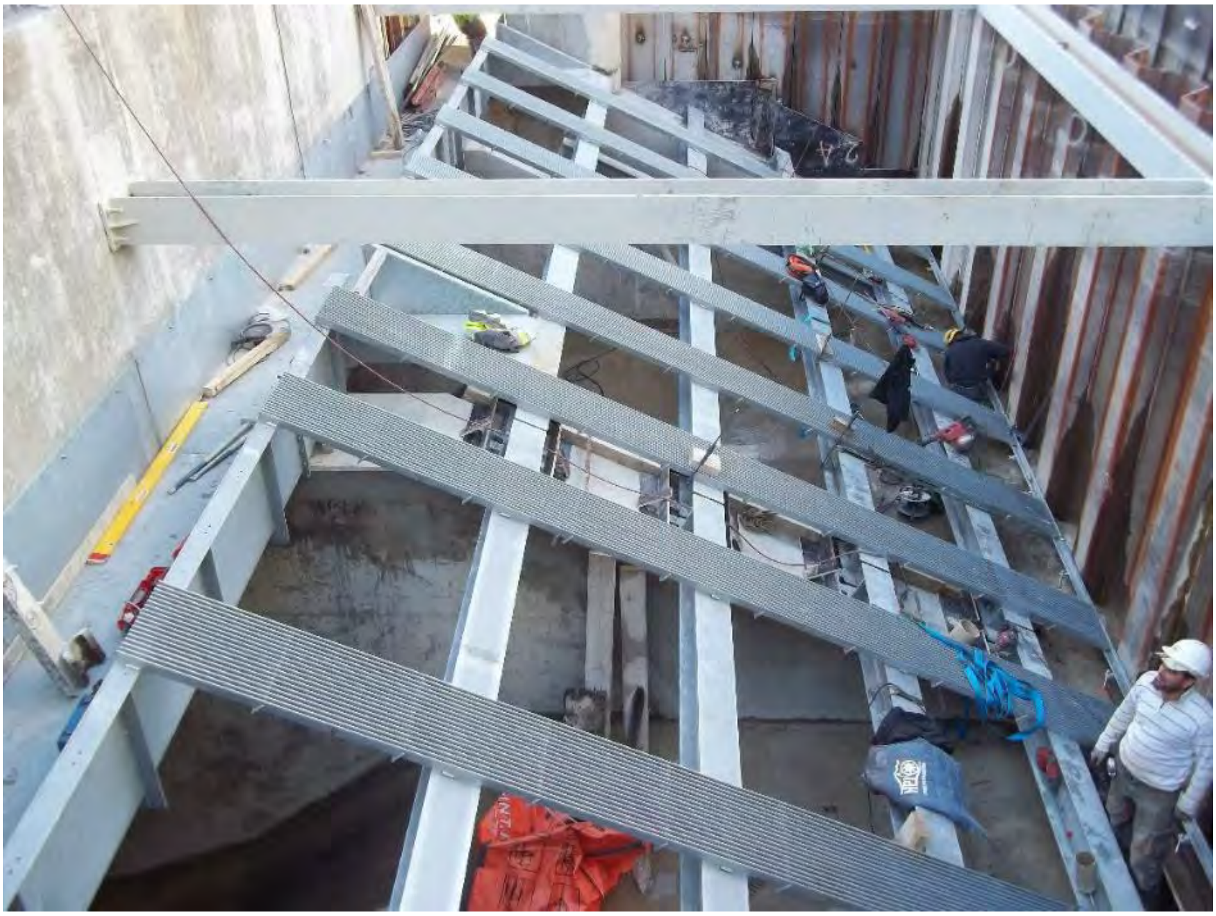


Vertical screen at an angle across the waterway.

In both cases, a side channel is provided for the fish to enter a bypass channel that then discharges back into the waterway downstream of the turbine intake. The water velocities are also specified.



V_a – water velocity
V_n – water velocity normal to screen
V_t – water velocity up the face of the screen
$V_t \geq 2V_n$ for an inclined screen
$V_t \geq V_n$ for vertical screens angled across the flow
$V_n \leq 0.5$ m/s in all cases



A sloping screen under construction, giving an example of the scale of the installation for a small hydro power plant.

In some cases, the environmental regulators require smaller gaps to keep fish from the turbine intakes. Screen gaps down to 6 mm have been successfully employed on hydro power plants on inland navigations. Traditional vertical bars and raking methods are not suitable and the bars are arranged horizontally. The screens are cleaned by wiping the face of the screens with a bristle or other resilient material from the upstream end to the downstream end, pushing the debris into chute that discharges downstream of the turbine intakes.

Horizontal bar screens with 6 mm gap

As the gaps between the bars become smaller, the blockage effect of the bars increases, and to maintain the flow area for the turbine, the size of the screens increases as a result. The head loss through the screen is not linear with the size of the gap. Smaller gaps have a larger head drop for the same flow velocity than larger gaps, so the flow areas are correspondingly larger for the same head loss. With small head differences available on inland navigations, the head loss through the screen can be a significant proportion of the head available. These all combine to make the screen areas potentially very large and, as a consequence, expensive to construct.

Gaps as low as 3 mm have been proposed by environmental regulators for some mini or micro hydro power schemes. With such small gaps, the size and cost of the screens can destroy the financial viability of the hydro power plant.

Large intake screens parallel to the main river flow on the Aire & Calder Navigation in England. These screens have the 6 mm bar spacing shown above and a horizontal automatic rake that pushes the debris into a debris chute. The picture illustrates the large size of the screens in relation to the hydro power plants output of 500 KW.

Fish friendly turbines

Fish can be damaged by passing through turbines by contact with the moving blades, being crushed in small clearances between moving parts or by the rapid changes in water pressure. To be considered fish friendly, a percentage of the fish passing through the turbine should survive.

Unfortunately, there is no international definition of a fish friendly turbine or how it should perform in relation to fish mortality. The definitions can be different for the same river passing through different countries.

The range of turbine operating head applicable to inland navigations usually leads to a choice of two types of turbine being employed:

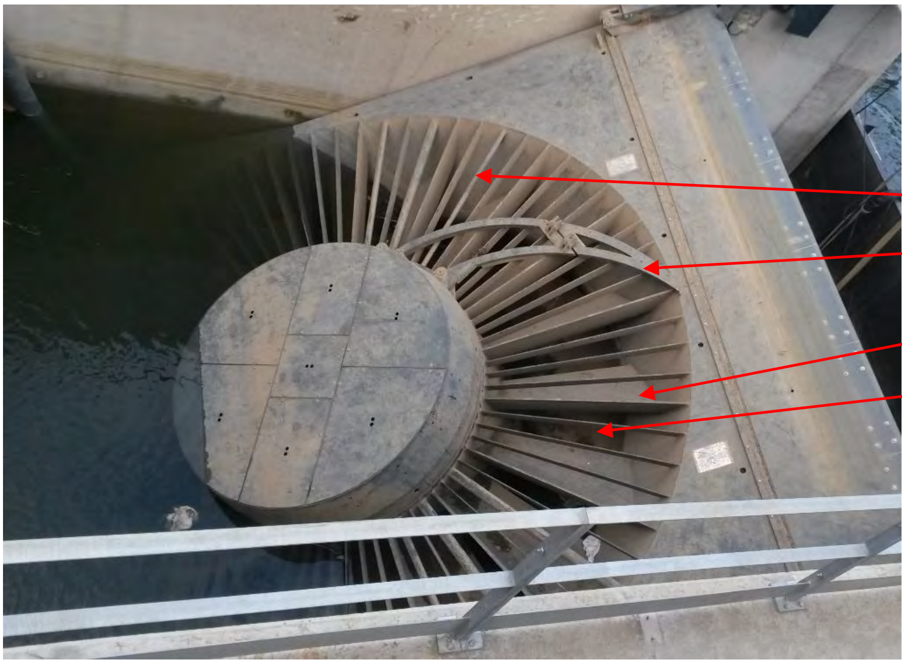
- Kaplan with variable or fixed pitched runner blades (propeller)
- Archimedean Screw

Kaplan turbines are more efficient energy converters and physically much smaller for the same power output than the Archimedean Screw turbines.

Various manufactures of turbines as well as the environmental regulators have done research on how fish friendly each type is. There is a considerable difference of opinion amongst the environmental regulators as to which turbines are fish friendly. However, Archimedean screws are considered to be fish friendly by environmental regulators throughout Europe.

Kaplan turbines are not usually considered to be fish friendly environmental regulators and their intakes must be screened. Depending on the type of screening required for a non-fish friendly turbine, the economics of overall scheme may favour a more expensive fish friendly turbine whose cost is offset by reductions in cost of the intake screens.

One variant of the Kaplan turbine the Very Low Head turbine, is considered to be fish friendly and widely used with 50 mm screen bar spacing upstream. This turbine has an applicable head range from 1.5 m to 3.4 m although this can be extended to 4.5 m.



Very Low Head turbine employed in the River Meuse in France showing the

turbine inlet screen

screen cleaner

the fixed guide vanes

the turbine runner

Floating Debris

Floating debris is a problem for all inland navigations whether the debris comes from vessels navigating the waterway, from trees on the banks or items dumped in the navigation. Traditionally this debris has collected on weirs or navigations locks and been flushed downstream during operation or in times of flood. Weirs with gates are often designed to allow the gates to be lifted clear to flush debris downstream.

The need to protect the intakes of hydro power stations from floating debris by the use of the screens will lead to a build-up of debris against the screens. The debris can either be lifted out of the waterway or left in the waterway and routed around the turbine intake and discharged into the downstream water course. Owners and operators will already be familiar with the type and quantity of debris that occurs in their waterways and can use the knowledge to drive the way in which debris is dealt with.

The turbine control system will need to respond to head differences across the screens and reduce or even stop the turbine flow if the screens become blocked. Automatic raking machines can be used to keep the screens cleaned but these are expensive to purchase and require maintenance. Shutting the turbines down and allowing the natural flow of water to take the debris away can be effective as can manual raking at times of high debris flow.

Sediment transportation and redistribution

On navigation waterways, a small head hydropower plant can benefit from specific hydraulic conditions head created by other hydraulic structures built for waterway management purposes:

- Small head hydropower station to generate energy
- A weir or drain to control the water level in the system
- A fish passage system, to allow fish migration both upstream and downstream of the systems
- Navigation locks to manage the ships to pass through the system.

Especially for water level maintenance in a system, the link between the hydropower station and the weir or gate (fish passage) construction is important for water level management. The combined water discharge from a weir/drain/lock and a hydropower station should match the average system discharge. In this way, the water level in the system and all other purposes of the river can be maintained, e.g.

- Sufficient water to allow for navigation
- Sufficient water in the river for drinking water extraction
- Sufficient water for adjacent industrial processes and irrigation (agricultural purposes)
- Maintainable water level in order to avoid flooding's

The hydropower plant main characteristics (head and discharge range) have thus to be defined regarding the whole system requirements and features, and in particular the discharge balance. It is common to see hydropower plants on navigation waterways able to operate in both pump and turbine modes. As such, in periods of excess discharge, energy is produced while, in low flow periods, loss of water in upstream reaches because of locks operation is compensate by pumping.

In order to calculate the upstream water levels from the hydraulic structures, backwater curves can be calculated. These can be calculated both analytical and by means of numerical simulations.

Extreme events

During extreme events, water systems should be able to discharge the high-water discharge peaks in the system. This is generally done by means of weirs or adjustable draining channels, as the small head hydropower systems have a limited capacity in comparison to the flood flows. At specific locations, where the hydropower system is blocking a part of the waterway, these systems can be adjusted (hydropower stations with adjustable discharge levels), often or even lifted, in order to avoid any influence on the discharge of high water peaks. In some cases it is necessary to be able to react on a short term bases.

When a small head hydropower system is located in an artificial branch of the waterway, the influence on the water discharge of the system is generally limited. In such a case, the hydropower plant can remain in operation during an extreme event.

In case hydropower schemes are not able to be removed from a water system or cannot be adjusted, upstream flooding's might occur, or the small head hydropower system might be flooded.

Small scale

Near small scale power stations, small scale effects might be present. For most situations, a scheme consists of a weir, a hydropower station, navigation locks and fish passages. All these hydraulic structures have intake and outfall systems, which cause jets that might influence each other. In this section, more information is given on typical situations that might be present.

For an optimal operation of the hydropower station, little head losses upstream and downstream of the system should be present. For systems located in the waterway, this is generally the case, but for systems that are located in a side branch additional resistance can cause head losses, which affect the energy production. For this reason, the full hydraulic system together should be in harmony and controllable.

Detailed flow patterns

To have an optimal design of the hydropower plant, the approach flow towards the turbines should be optimal, with minimum hydraulic losses. Sharp bends or rapid changes in the flow should be avoided. To investigate the hydraulic losses upstream and downstream of the turbines, both numerical and physical modelling could be applied.

Impact on navigation

Hydrodynamic forces on a ship can be significant near small head hydropower stations, due to strong concentrated flows near the outflow of a power station, in particular for recreational and smaller ships.

The hydrodynamic forces exerted on the ship's hull result from all the water movements and pressure variations (e.g. pressure force due to differences in water level, velocity changes due to jet flows, drag forces caused by flow around the hull, turbulence force due to the energy dissipation in the lock chamber). The resultant sum of all hydrodynamic forces on the hull can be termed the 'ship force'.

These are the forces measured on the physical model, with two components: one longitudinal and one transversal. Their numerical prediction requires a sophisticated 3-D numerical model able to reproduce the movement of the ship, which means long and costly numerical calculations.

Morphology

Hydraulic structures have an impact on the waterway or channel flows. Weirs, locks and hydropower schemes can only transport limited amounts of sediment. If the sediment cannot be transported through a system, it might influence the approach flow patterns of a system. To investigate potential sediment accumulation near the intake of a power plant, numerical modelling is often applied, incorporating significant upstream and downstream regions, to model the total sediment balance of the system. Detailed nearfield models, incorporating all turbulent 3D flow patterns could indicated dead zones, which are susceptible for sediment accumulation.

Bed protection

Near intake and outfall systems of hydropower plants, flow acceleration is present, that might cause erosion near the structure. For this reason, bed protection is often applied, as erosion pits near a hydraulic structure can undermine the hydraulic structure. To design bed protection near hydraulic structures different guidelines exist, such as the Rock Manual, [CUR, 2007]. If no precautions are taken, the bed needs to be monitored and if necessary maintained (dredging, equalising, refilling).

Conclusions

Dealing with environmental regulators is an important aspect of constructing a hydro power plant on an inland navigation. The regulations vary from country to country and even from region to region. However, they are essentially related to the elements contained in this paper and they can, usually, be dealt with by design and planning in the initial stages of the project.

The owners and operators can view the environmental regulations and restraints as a necessary evil, or a more positive view can be taken where the environmental benefits and improvements are used as part of the justification for the scheme, possibly enabling environmental grants to be applied for and good will to be generated in addition to generating green energy.

WG197 SMALL HYDROPOWER IN WATERWAYS BEST PRACTICE

WG197

Introduction

Many inland navigations have the basic ingredients for a hydro power plant in existing infrastructure. A number of inland navigation owners have taken advantage of their infrastructure to build hydro plants and to generate extra revenue or reduce operational costs. Whenever the infrastructure around rivers or canals is modified, improved or even just maintained, many factors beyond simply doing the work come into play. Some of these can add considerably to the cost or even stop a project before it starts.

This paper briefly describes the issues and then looks at how these can be addressed with direct reference to plants that have been constructed on inland navigations. Generalised hydro plants are not considered, only those built specifically on inland navigations are included.

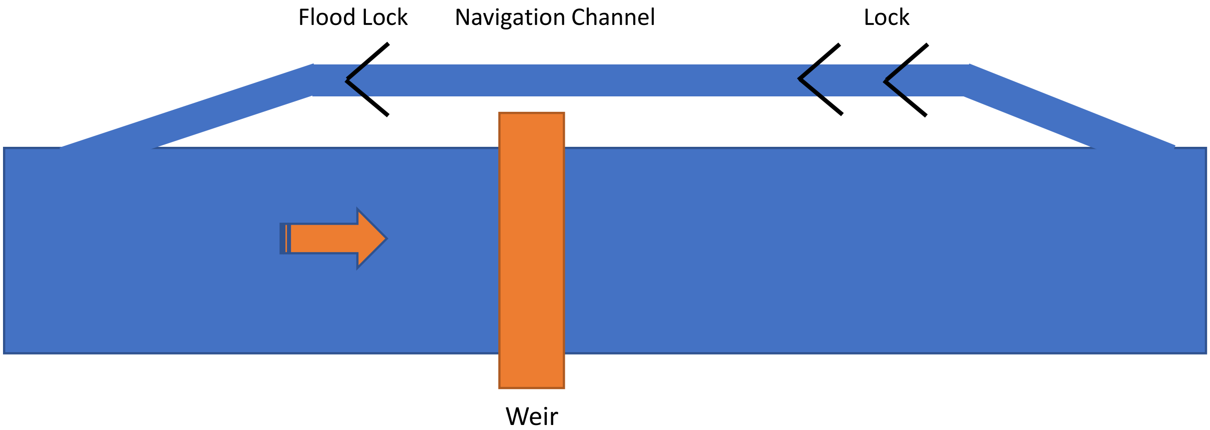
The ways in which these owners and operators have dealt with the issues provide a useful insight into what can be achieved.

Hydro-power in inland navigations

A typical arrangement is shown below with a navigation channel around a weir. The navigation might be a river navigation or a completely man-made canal. The flows might be less in the case of a canal, but they can still be significant and still capable of delivering meaningful energy outputs.

All the basic ingredients for a hydro power station are present:

- Impounded water
- A difference in levels – a head
- A flow of water



The owners, managers and operators of inland navigations all know the difference in levels and have a historical record of the variation of flow over time at the at each weir and lock location. Whilst the water has other uses, particularly for navigation and compensation flows, any surplus can be put to good use generating power.

The technology of hydro power is widely understood and generally mature, but manufacturers have recently been developing turbines to operate at lower heads, sometimes combined with variable speed generators, to exploit more marginal heads and flows.

The working group has restricted itself to a head range of 1.5 m to 15 m and a power range from 100 kW to 10 MW as might be encountered on inland navigations.

Issues for development

Navigation

The prime function of inland navigations is, of course, navigation. The development of a hydro plant and the se of water for generation needs to be done in such a way that navigation water levels and water supply are not affected. Priority is always given to navigation water supply and level control for navigation.

Of the two main water requirements for navigation, level control can be problematic when a significant portion of the flow in the waterway is being diverted through the turbines. When changes of flow through the turbines occur quickly, the changes to level can go beyond the navigation limits.

During starting of the turbines, the flow change is gradual and regulated by the control system specifically not to affect the navigation levels. The same can be applied to a controlled stop. However, when an emergency stop is required, perhaps because of an external influence, the turbine flow will be stopped very quickly. The water that was flowing through the turbine must go somewhere. Initially, a rise in the water level upstream of the turbine will occur. If the upstream entrance to the navigation channel is close to the weir, then the change in level can affect navigation unless corrective interventions are made. The further upstream the inlet to the navigation channel is from the weir, the less the problem becomes.

With a fixed weir, the problem can only be dealt with by modelling the flows and setting the maximum flow that the hydro plant can take to remove the possibility of affecting the navigation levels on emergency shutdown. Where the weir has an element of active control such as automatic gates or an inflatable rubber weir, then studies need to be done into the reaction of the existing control system to an emergency turbine trip – is it fast enough to keep the level change within acceptable limits – and if not, links will need to be established between the hydro turbine operating system and the weir level control system to make the system respond quicker in the event of a turbine emergency shutdown.

Flood Control

Many navigations are based around rivers and seasonal flooding has occurred over many years of operation. Ways of dealing with the floods have been developed and implemented. The addition of a hydro power station must maintain the existing flood capacity and not make the situation worse.

Where space is available, building the hydro power station off line in a new channel is the obvious solution. Where this is not possible, and the hydro power station takes up part of the existing flood conveyance channel, other ways must be found to enable to pass the flood waters.

Fish, eels and environmental considerations

The construction of a new hydro power plant on a navigable waterway, whether that is a river navigation or a man-made canal, will require consideration of the effect on fish and particularly migratory fish. Even where existing weirs have fish pass provision, when a new hydro power station is proposed, a review of the whole installation as an obstruction to the passage of fish and eels will undoubtedly be required and additional fish pass provision may be required. Where there is no existing fish pass provision, a new one will usually be required as part of any hydro power development.

Examples of best practice

Lock 51 Saint James, Meuse River, France

Like many navigation structures on the Meuse River in France, Lock 51 had a stake weir to impound the water for navigation. To renew and modernise the site, a new weir with an inflatable rubber crest was installed to actively manage the upstream water level. A new channel was cut around the weir to enable a hydro power plant and new fish pass to be built.

Active control of the navigation level upstream of the weir using the inflatable rubber crest is done separately from control of the hydro power station, but when an emergency shutdown of the turbines occurs, a connection between the two control systems ensures that the inflatable weir responds quickly enough to keep the level within the navigation limits. This is a particular challenge when the flow through the turbines represents 80% of the flow in the river which it does at certain times of the year.

Lock 51 Saint Joseph. New channel for hydro power and fish pass on the left with the existing stake weir still in place during the construction phase.

Lock 51 Stain Joseph. Fish pass of the double vertical slot type with bed substrate

There was no fish pass around the existing stake weir and the construction of the new hydro power plant required the fish pass to be built. A pool type with double vertical slot and bed substrate was chosen. This type is widely used and suitable for many fish species including benthic (bottom dwelling) fish that use the bed substrate to climb up the difference in levels.

The turbines used are “Very Low Head” turbines, a development of the Kaplan type of turbine with large slow-moving blades that are classes as being fish friendly. Only the Very Low Head turbine is considered to be fish friendly by the environmental regulators in France. Other variants of the Kaplan turbine are not considered to be fish friendly. The blades have a variable pitch which is combined with a variable speed drive to enable a wide range of flow conditions to be utilised efficiently at low heads in the range 1.5 m to 3.4 m.

Lock 57 Ham, Meuse River, France

This installation is similar to Lock 51 on the same river with the exception that the hydro plant was constructed not in a new channel, but in an existing channel that conveys flood water. The Very Low Head turbines can be lowered out of the way and protected by steel doors to allow the floods to pass over the top. A similar pool type fish pass with twin vertical slots was constructed as part of the development.

Lock 57 Ham. Inflatable rubber weir in operation. The original stake weir has been removed. It was further upstream of the new weir and its position is marked by the piers still visible.

Lock 57, Ham. Very Low Head turbines in operation. In times of flood, the turbines are lowered down and the doors are closed over the turbines to allow the full area to be used for flood discharge

Lock 1 Hastiere River Mueuse, Belgium

Downstream from Lock 57 on the same river, this time in Belgium, Lock 1 at Hastiere presents a different set of challenges. The river flows through a comparatively narrow valley with major road and railway infrastructure on one side and a relatively steep, wooded slope on the other. The weir consists of three radial gates that function both to manage navigation level and pass flood waters by rotating to be fully above the flood water level. The upstream entrance to the navigation channel is close to the weir.

There is no space on either side of the weir to construct a new channel for a hydro power plant at a reasonable cost. However, with 100 m^3/s of water available for generation, the potential energy output could not be ignored. A unique solution was developed to utilise the water whilst not interfering with the flood capacity of the weir.

Six propeller turbines with fixed inlets and fixed runner blades are mounted on a steel deck that is installed downstream of one of the three weir radial gates. In operation, the gate is lifted clear to allow an uninterrupted flow into the turbines. The turbines are switched on as required to utilise the water available.

In times of flood, the whole assembly of turbines is lifted vertically upwards to clear the gate and allow it to fulfil its normal function.

The weir has an existing fish pass and no additional provision was made as a result of installing the hydro power plant. The turbines employed are classed as fish friendly in Belgium.

Lock 1 Hastiere showing six turbines mounted on a steel deck immediately in downstream of the weir gates. The vertical tower is part of the lifting equipment that lifts the deck clear of the river in times of flood.

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Electromécanique
HERSTAL

Lock 1 Hastiere showing six turbines raised up above the water to allow unrestricted flood passage.

Knottingley and Kirkthorpe, Aire & Calder Navigation, UK

These two hydro power stations constructed around fixed weirs on the Aire & Calder Navigation in the UK. Both use double regulated Kaplan type turbines with variable inlet guide vanes and adjustable runner blades. There are two units at Kirkthorpe and a single unit at Knottingley. Fish and eel migration were major factors in the design of the two stations. Kaplan turbines are not considered to fish friendly by the UK Environment Agency. Exclusion of the fish from the turbines at the inlet was considered to be the only acceptable solution.

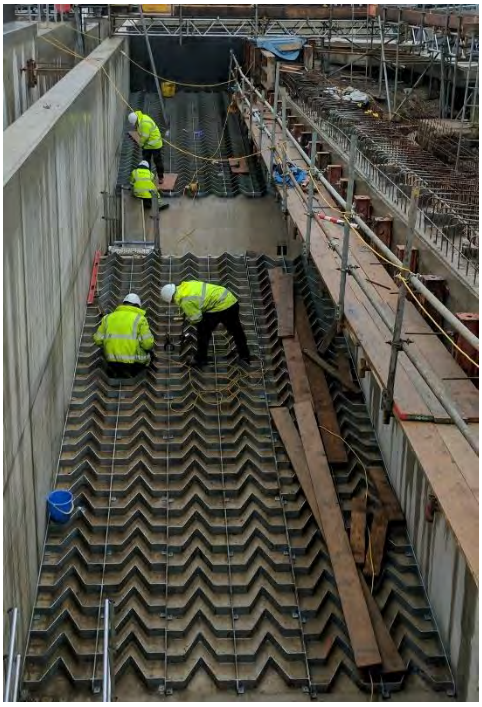
The UK Environment Agency required that the inlets be screened down to 6 mm and 9 mm at Knottingley and Kirkthorpe respectively. This has resulted in large inlet screens with horizontal bars rather than the more normal vertical screen bars that are wiped horizontally rather than being raked vertically in the traditional way. The large size was necessary to keep the head loss across the screens low. With only a few metres of head available, the head losses across the screens can reduce the energy produced significantly.

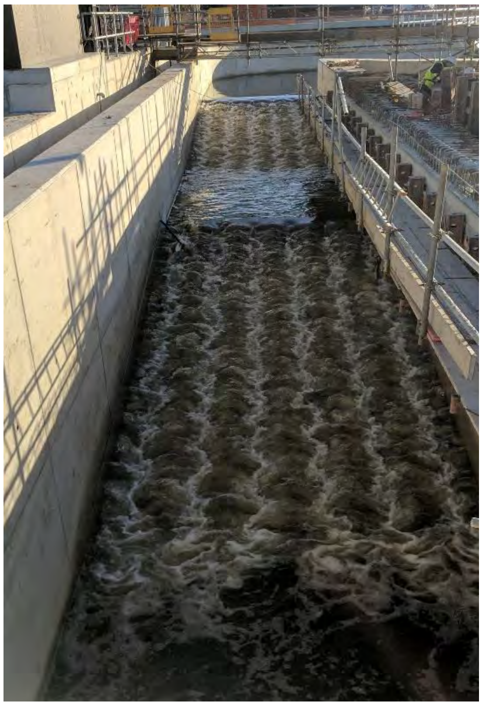
The inlets are parallel to the river and the wiper runs from the upstream end to the downstream end only. The debris is pushed towards a debris chute that takes it around the weir and into the river below the turbines.

Outlet screens are also provided to prevent upstream migrating fish from entering the turbine, but these are not raked.

Knottingley Hydro Plant, Aire & Calder Navigation. Showing the large intakes necessary and the screen wiper at the upstream end.

Both installations required new fish passes and eel passes to be constructed. The eel passes consist of pipes with an internal mat that has a constant flow of water trickling over it. The fish passes are of the Larinier super-active type with a series of steel baffles installed in a channel seen below under construction and in use – pictures from Yorkshire Hydropower.





Lock Kwaadmecheien – Ham, Albert Kanal, Belgium

Unlike the previous examples, the Albert Kanal is an entirely man-made waterway. It is fed by a number of rivers, including the Meuse/Maas River. Most of the year there is excess water but at certain times there are water shortages which can affect navigation. To overcome the shortages, pumping stations have been constructed at some of the locks using large Archimedean screw pumps with a capacity of 5 m^3/s. There are plans to equip all the locks on the canal with similar pumping stations in the next few years.

These pumps can be reversed and act as turbines to generate power when there is excess water available. The availability of water for generation is determined entirely by the needs of navigation.

The Archimedean screws whether operating as a pump or turbine are classes as fish friendly.

Conclusions

The existing infrastructure associated with inland navigations is well suited to the addition of hydro power plants of mini and small size. The resulting energy can either be used by the operator of the navigation to reduce expenditure or sold to provide additional revenue.

There are many challenges which vary from country to country but the examples presented show what can be achieved.

BUILDING A DECISION SUPPORT SYSTEM FOR THE TERNEUZEN LOCKS : COMBINING OPTIMAL MANAGEMENT FOR WATER AND SHIPPING

by

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ABSTRACT

The Terneuzen lock Complex in the Dutch city of Terneuzen gives access to the Ghent-Terneuzen Canal and thus to the port of Ghent (Belgium) from the Western Scheldt. Currently, the construction of a new lock is being prepared, which is expected to be operational in 2022.

In the context of the licensing process for the new lock, how to deal with the available (fresh)water in the most optimal way was examined. The quality values for water (mostly focusing on salt intrusion) must be met, together with the shipping requests and taking into account the daily water management issues of the Canal. Following the conclusions of an expert group, set up to optimize the use of the lock complex, the decision has been made to prepare for a Decision Support System for the Ghent-Terneuzen Canal, in a joint Flemish-Dutch collaboration. This DSS will provide the planners and operators with streamlined and uniform information and support them in the economic and ecologic optimisation of the daily operations of the lock complex.

The paper gives an overview of the background and the preparation phase of the DSS.

1. INTRODUCTION

1.1 The Ghent-Terneuzen Canal and the Terneuzen Locks

The Ghent-Terneuzen Canal is situated on Belgian and Dutch territory and was constructed between 1823 and 1825. It links the Harbours of Ghent and Terneuzen (North Sea Port) to the Western Scheldt and thus to the sea.

Figure 1 : The Ghent-Terneuzen Canal (in red), with the Western Scheldt and the North Sea (Google Maps)

The Canal ends, through the locks at Terneuzen, in the Western Scheldt as can be seen in Figure 1. The Western Scheldt is the mouth of the river Scheldt, and forms part of the Scheldt estuary. The Scheldt estuary has an average tidal range of 4 meters; and through its open connection with the North Sea it

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has a gradual transition from seawater to freshwater. The salinity concentration near Terneuzen is on average between 10 and 15 g Cl-/L. The upstream border of the Canal, as mentioned, is the complex at Evergem. This consists of two segment weirs and two locks, to ensure the locking of inland vessels sailing between the Western Scheldt, the port of Ghent, the coastal harbours of Ostend/Bruges, and France via the rivers Lys en Upper-Scheldt. The first lock came into service in 1965. It is 16 m wide and 136 m long. The second lock, delivered in 2009, was constructed to accommodate inland barges up to 4400 tons, the adopted class for the new Seine-Scheldt link. Every year the locks are passed by about 30.000 vessels. The weirs deliver input of freshwater in the Ghent-Terneuzen Canal.

The current lock complex at Terneuzen consists of three locks. The oldest, called Middensluis, was opened in 1910, has a length of 140 meters and a width of 18 meters. The lock was renovated in 1986. In 1968 two new locks were opened. The Oostsluis or inland lock, which has a length of 280 meters and a width of 23 meters. The Westsluis or sea lock, which has a length of 290 meters and a width of 40 meters. It is suitable for seagoing vessels with a draught of up to 12.5 meters and a load capacity of 83,000 tonnes. At this lock, provisions have been made to prevent salt water from entering the Canal. Every year the locks are passed by about 10,000 seagoing vessels, more than 50,000 inland barges, and 3,000 recreational vessels.

Figure 2 shows the geographical situation of the Canal, and the location of the upstream locks at Evergem (Belgium) and the downstream locks at Terneuzen (Netherlands).

Figure 2 : schematic representation of the Ghent-Terneuzen Canal (Deltares, 2017)

The Canal has a total length of approximately 32 kilometres. Today, the water depth of the Canal is 13.50 m and the Canal is therefore accessible for ships up to 125,000 tons. The maximum dimensions of seagoing vessels are 265 * 34 * 12.50 meters. On Dutch territory the width at the bottom is 62 meters and at the water level 150 meters. In Flanders these dimensions are 67.7 meters and 200 meters, respectively.

A 1960 treaty between the two countries, with amendments in 1985, regulates the agreements on the water level, the supply of freshwater upstream and the available discharge capacity and minimization of the salt supply at the lock downstream.

The supply of freshwater to the Canal is controlled via the weir and locks at Evergem (Belgium), where water from Leie (Lys) and Schelde (Scheldt) is transported via the Ringvaart to the canal. Figure 3 gives the discharge distribution at the Evergem Weir. In addition to this, limited quantities of fresh water are also supplied via the Moervaart and the Avrijevaart (not on the map), which drain into the Canal. In addition to the shipping functionality, the canal is also important for the discharge of high waters and preventing flooding upstream. The average total yearly discharge is 24 m3/s, with maximum values reaching up to 150 m3/s in periods of high discharges from Lys and Scheldt (De Boeck et al, 2012) . During periods of high discharges, and depending on the intensity of the sluicing, the lock operations at Terneuzen can be suspended and shipping on the canal can be halted. During periods of low discharges and when the combined discharge of Lys and Scheldt are very low (order of magnitude 10-20 m3/s) the total discharge into the canal can be close to 0 m3/s.

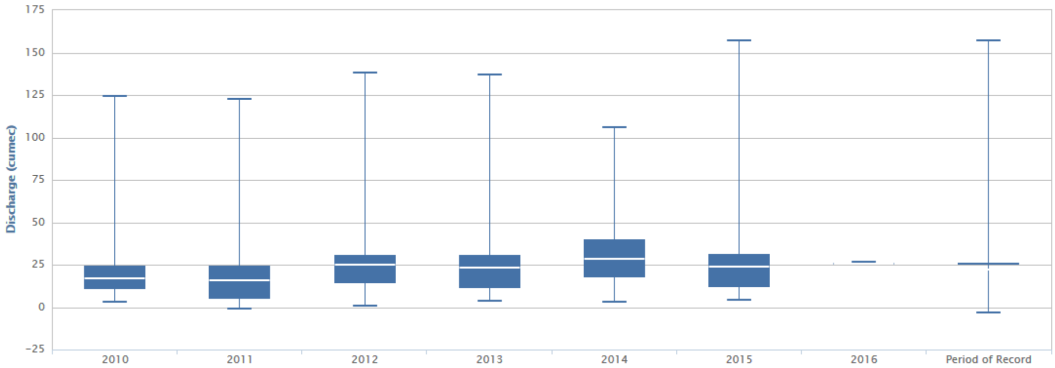

Figure 3 : discharge distribution at the Evergem weir (From waterinfo.be)

The water in the canal is brackish and is connected to the saline Western Scheldt through the locks in Terneuzen. The salinity in the Canal decreases in the upstream direction, and is highly dependent on the freshwater discharge as can be seen in Figure 4 and Figure 5 : during summer salinity increases and the salinity wedge moves in certain cases up to the beginning of the Canal in Ghent.

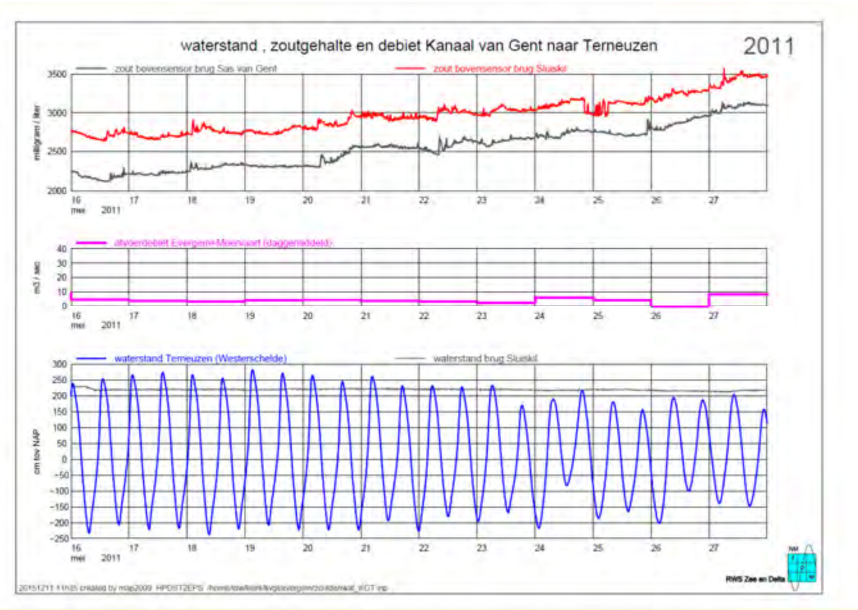

Figure 4 : Development of the chloride content at Sas van Gent and Sluiskil at a low freshwater supply (<10 m3 / s) to the canal. Water loss through lockage at the current complex is approximately the same or less than the supply to the canal. The result is a decreasing water level (Sluiskil) and rising chloride levels as a result of the salt load from the Western Scheldt. The ratio between the chloride content at Sas van Gent and Sluiskil is a fairly constant factor (around 0.8 to 0.9). (From Waterinfo.rws.nl)

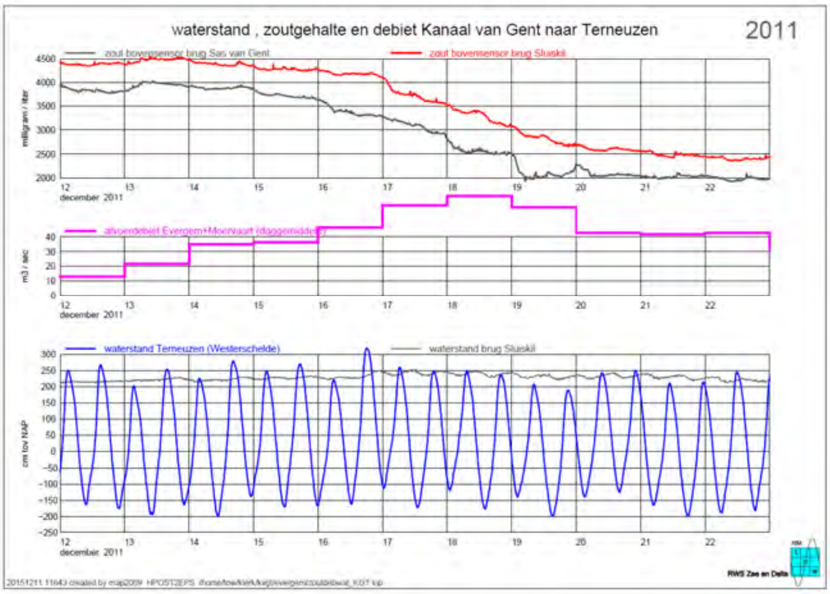

Figure 5 : Development of the chloride content at Sas van Gent and Sluiskil at a strongly increasing freshwater supply on the Canal. In this situation, there will high freshwater discharges. The result is an increasing water level and decreasing chloride content as a result of flushing the Canal (from waterinfo.rws.nl)

1.2 The New Lock at Terneuzen

In 2004, joint Dutch-Flemish research into the maritime accessibility of the Ghent-Terneuzen Canal was started. This research showed that the development potential of the Canal as a waterway was in the long run hampered by the dimensions, robustness and capacity of the current locks at Terneuzen. Further research in the subsequent years focused on the different possible solutions and in 2012 The Netherlands and Flanders agreed on building of new lock at Terneuzen. With the realization of the New Lock larger seagoing vessels can sail to the port of Ghent through the Ghent-Terneuzen Canal. The capacity of the lock complex also increases, which reduces the waiting time for inland vessels.

In 2017 the study phase and the permit process were completed and the construction of the New Lock was started. The lock is expected to be completed in 2022. The New Lock at Terneuzen is a Flemish-Dutch project, carried out by the Flemish-Dutch Scheldt Commission.

The lock will be 427 meters long, 55 meters wide and 16 meters deep and is shown in Figure 6. The dimensions correspond to those of the new locks of the Panama Canal.

Discharge and levelling is done through the lock gates. No infrastructural measures for salinity intrusion are built into the lock, but no-regret measures are taken during construction to be able to implement those later on, when necessary.

Figure 6 : sketch of the lock complex at Terneuzen. The New Lock will be constructed centrally in the complex and will replace the original Middensluis (VNSC, 2015)

1.3 Legal Framework and EIA process for the New Lock at Terneuzen

The project must comply with the applicable regulations, legal and administrative frameworks. For water management relevant frameworks are the Water Framework Directive, which for the Dutch part of the Canal stipulates that the three years averaged salinity should be between 300 and 3000 mg/l. It should be noted that the WFD salinity requirements only apply to the Dutch part of the Canal; the Flemish part does not impose any requirements on salinity. The 1960 treaty (Belgium and the Netherlands, 1960), defines the minimum and maximum water levels in the Canal and sets these at 2,13 NP +/- 25 cm. The treaty also contained agreements on the minimal amount of freshwater discharge, the measures against salinity intrusion and the available discharge capacity downstream.

The construction of the New Lock follows the Dutch procedures and an Environmental Impact Assessment was carried out.

For salinity, the EIA (VNSC, 2015) concluded that until 2050, the averaged salinity is expected to be within the WFD standard, also when the average discharge on the canal would decrease in summer, due to climate change. Permanent measurements and follow up are however essential to monitor the results of the EIA. It was noted that differences between individual years can be significant and that in individual years the salinity could be higher than 3000 mg/l. Exceeding the (3 year averaged) WFD salinity standard could lead to a (temporal) shutting of the lock procedures in order to minimize salt intrusion through the lock. Because of this, a number of trajectories have already been started in order to be optimally prepared for the future. No-regret measures are already being taken during the construction of the New Lock, to facilitate and simplify the implementation of salt separation measures in the future.

For water management, the EIA concluded that both high and low discharges will also lead to a limitation of lock operations in the future. The impact of the high water discharges on the operation of the New Lock will be limited (as discharge will take place mainly through the other 2 locks in the complex). But for low water discharges, the EIA calculated that locking operations in the New Lock would be halted for 4,3% of the time in order protect the water level in the canal from water loss through lockage.

In the longer term, the challenge to meet the prerequisites for water quantity and quality will increase. The EIA indicates that this will be after 2050, but this could also occur earlier, with an increase in shipping traffic and the frequent occurrence of long periods with low river discharge from Scheldt and Lys. The most critical are the situations where, as a result of prolonged low upstream supply from Evergem, there is both a low water level and a threat of exceeding the WFD standard. At those times shipping to and from the canal will be seriously hindered by the fact that on the one hand the operational minimal water level cannot be guaranteed, and on the other hand because the number of locking operations must be limited in order to counter the increasing salt load on the canal.

2. **THE DECISION SUPPORT SYSTEM : TWO-PHASED APPROACH**

Following the conclusions of the EIA study, a Flemish-Dutch group of experts was asked for guidelines to optimize the lock operations in function of minimum waiting times, optimal discharge planning and controlled salt intrusion. In the first instance the expert group focused on salinity, but given the issues concerning high discharges and low freshwater availability, and the necessary coordination needed for this between Flanders and the Netherlands, it was decided to take a broader approach in order to arrive at a number of guidelines for the operational management of the entire lock complex.

The final guidelines aim to:

- Respect the minimal and maximal canal level, as stipulated in the 1960 treaty
- Respect the norm for salinity, with respect to the WFD
- Ensure that ships pass through the complex as quickly as possible, with minimal waiting time

The expert group also came to the conclusion that, in order to be able to apply these agreed guidelines properly, a decision support system (DSS) would be a welcome aide. This DSS checks the boundary conditions for water management with the requests for shipping and delivers all information to the operator so that an objective choice for the optimal use of the lock complex can be made. In addition to this, the DSS also gives feedback to the operator on the consequences of the chosen lock operations on water quality (salinity) and water levels in the Canal.

The following paragraphs first zoom in on the work of the expert group, where the guidelines were drawn up; then look in more detail at technical preparation for the decision support system.

2.1 Expert group (2015-2016): guidelines

The expert group included representatives from both countries, from the water management, the lock management as well as the port authorities. The expert group focused on three scenarios, in addition to the standard use of the locks: (i) periods of high upstream discharge, (ii) periods of low upstream discharge and (iii) periods of high chloride content, as well as the possible combinations thereof. The working group has agreed on limit values for each of the possible scenarios to which actions are linked to ensure that the applicable boundary conditions for shipping and environment can be respected. For each of the three scenarios (high discharge, low water levels in the Canal and high salinity) management options were given and longer-term gradual measures were defined, with minimal disruption to shipping; in order to avoid that lock operations should experience an acute shut down because e.g. the salinity levels in the Canal were too high or the water level in the Canal too low. The expert group looked for a good balance between sufficient and timely intervention on the one hand and minimizing nuisance as much as possible on the other. Early intervention can lead to unnecessary delays for the vessels. To intervene too late, for example with regard to the underestimation of the minimum level, can lead to very long periods where only limited lockage is possible. The aim therefore was to provide guidance for pro-active water and navigational management, based on the relevant variables.

In general, this meant that for each of the three scenarios the expert group considered over which period certain processes can be allowed to comply with the prerequisites. This was done taking into account the boundary conditions and the relevant time period (for high water, this is obviously shorter than for a too high chloride content which reacts much more slowly). For example, in the event of low water levels in the Canal, it will be calculated how many of which type of lockage, with a certain amount of lockage water, can still be admitted over a certain period in order to avoid total blockage of the entire complex. The different lockage operations can be compared to building blocks; and these building blocks can in turn be used in different combinations to build an operational concept.

<u>High discharge scenario</u>

When high discharges occur, coming from upstream Lys and Scheldt, one or more locks need to be used for discharging and no ships can pass the lock. Modelling studie were carried out in the framework of the EIA to estimate the percentage of time that locks are blocked. Results showed that the total complex is blocked at 0 % of the time (i.e. there will always be a lock that is accessible to shipping), but individual locks can be blocked, so a careful planning is needed to minimize the impact of the high discharges on shipping.

The following measures can be taken

- Regulate the upstream discharge at the complex in Evergem (within the boundaries of flood safety)
- Preventive discharge through the locks in Terneuzen until a minimal water level of 2;05 mNAP in the Canal;
- Discharge through the complex in Terneuzen
    - Westsluis : via 2 circulation sewers
    - Oostsluis : via the lock gates
    - New Lock Terneuzen : via the lock gates

Year	Middensluis	Oostsluis	Westsluis	New Lock
current	2,1 %	0,0 %	0,5 %	-
2030	-	0,4%	3,2%	0,0%

Table 1 : percentage of time the lock complex is not accessible for ships due to high water discharges (VNSC, 2015b)

The decision whether there is high discharge scenario is taken based on the predicted discharges for the upcoming 48 hours. These discharges for the upcoming 48 hours are indicative, based on the expected discharge from Lys and Scheldt. It is possible to lower the water level in the canal, in the prospect of high discharges. This can be done 24 hours before an announced high discharge, taking into account the minimum defined water level and taking into account the seagoing traffic that is expected during that period. These discharges should take place when no hindrance for shipping is expected.

If the predictions indicate that the level will be above 2.38 mNAP within 48 hours if no action would be taken, then the high water scenario is activated.

Based on the predicted discharges and water levels on the canal, it is then calculated how many m3 is to be discharged over the next 48 hours. Taking into account the expected shipping traffic and the predicted water levels on the Western Scheldt (including tidal cycles), the provisional discharge targets for the next 24 and 12 hours are calculated. Based on these preliminary targets, the operator will check how the required discharge quantity for the next 24 and 12 hour period can be achieved with minimal hindrance for shipping. The assignment of seagoing vessels to a lock passage is done 6 hours in advance; the allocation of inland vessels to a lock passage is made 3 hours in advance. Within this period of six hours, the operator has only limited flexibility to adjust the discharge regime in view of the agreements with shipping.

The combination of the available information results in a preliminary distribution of the discharge over the different locks in the complex. The targets are updated regularly and tested against the expected shipping traffic.

In consultation with the sluice operation upstream in Evergem, the discharge target is further refined and a definitive discharge targets for a 3 and 6 hours periods are set.These targets are binding, which means that at that moment shipping is subordinate to the necessary discharge.

Low water level scenario

The water level in the Canal is regulated by the upstream freshwater discharge in Evergem and the downstream discharge in the Western Scheldt through the Terneuzen Lock complex. When the discharge through lockage water is higher than the upstream discharge, there is a nett loss and water level in the Canal will fall. The minimal water level in the Canal is agreed at 1,88 mNAP. However the larger sea-going vessels that access the harbour of Ghent need a minimal water level of 2 mNAP. Per nett 1 m3 / s loss per day, the water level on the Canal decreases by 1 cm per day. It is assumed that the water loss from lockage in the Terneuzen complex, including the New Lock, is on average 12 m3 /s with regular use of 3 locks and an average shipping intensity; during summertime the freshwater discharge can be as low as 5 m3/s. Research carried out in the framework of the EIA study showed that in 2030, the New Lock will be not accessible during 4,7 % of the time because of low water levels in the Canal.

Climate	Middensluis	Oostsluis	Westsluis	New Lock
current	0,0 %	0,0%	0,0%	-
2030	-	2,6%	0,8%	4,7%

Table 2 : percentage of time the lock complex is not accessible for ships due to low water levels (VNSC, 2015b)

The freshwater discharges are known up to 48 hours in advance. If the predictions show that the nett daily average loss of the next 48 hours will be more than 4m3/s for the same amount of lockage, then this scenario becomes active.

The following management measures are proposed (the order in which the measures are proposed is indicative, and can be adjusted by the operator according to the range of ships to be locked):

- Maximizing the freshwater discharge at Evergem (when available)
- Optimizing the lockage process (only passages with completely filled lock chambers);
- Only locking in the New Lock if the water level in Western Scheldt is the same or higher than in the Canal, causing no nett loss
- •Only locking in the Oostsluis if the water level in Western Scheldt is the same or higher than in the Canal, causing no nett loss
- Only locking in the Westsluis if the water level in Western Scheldt is the same or higher than in the Canal, causing no nett loss
- Depth restriction for the (sea) shipping in the Canal;
- Aim for a level of 2.30 mNAP in the period from 1 April to 31 October in so far as the impact to navigation (taking into account the height restrictions for bridges further upstream); and connected watercourses remains limited.

In addition to this, restrictions on the maximum allowed nett freshwater loss were defined.

Two options are possible: when prioritizing seagoing vessels (requiring a minimum depth of 2.00 NAP), the nett loss of water will be limited to 0 m3 / s from 1.95 mNAP. In prioritizing the levels from the treaty (1.88 mNAP), some loss will be allowed at that time.

Water level in the canal	**Management measures to reduce water loss and maintain water level in the Canal**
Higher than 2,30 mNAP	No limitations on lockage water
Between 2,20 mNAP and 2,30 mNAP	Maximum nett loss through lockage (next 24 h) **3.5 m^3/s**
between 2,13 mNAP and 2,20 mNAP	Maximum nett loss through lockage (next 24 h) **3 m^3/s**
between 1,95 mNAP and 2,13 mNAP	Maximum nett loss through lockage (next 24 h) **2 m^3/s**
between 1,88 mNAP and1,95 mNAP	*In accordance with the 1960 treaty* : **1 m^3/s**

	Taking into account seagoing ships : **0 m³/s**
Water level Canal below 1,88 mNAP	No loss through lockage allowed

Table 3 : overview of maximum allowed nett loss of freshwater, related to the water level in the canal

Comparable to the salt load, the available space (defined by this maximum permissible lockage loss per day) is built up from a combination of a number of "building blocks". These building blocks consist of lockage with a certain lock, at high water, low water and average water levels on the Western Scheldt, each with an associated nett loss of fresh water. Which of these building blocks are used is free for the operator to choose and decide on the basis of the types and intensity of shipping.

High summer-average chloride levels

The WFD standard stipulates that the three year summer average (from 1 April to 30 September) must be between 300-3000 mg / l at the measurement point at Sas van Gent. This means that controlling the chloride content in the Canal requires an especially long-term planning.

If the forecasts of the triennial average, calculated at the end of the summer and based on the projection based on the current chloride content, show that the three-yearly average chloride content is higher than the permitted chlorides standard, this scenario becomes active.

The following management measures are implemented:

- Maximizing the freshwater discharge at Evergem (when available)
- Optimizing the lockage process (only passages with completely filled lock chambers);
    - Use the New Lock for the larger ships, taking into account the available salt balance. A possibility to limit the salt load is to restrict use around high water.
    - Use Oostsluis for shipping, taking into account the available salt balance. A possibility to limit the salt load is to restrict use around high water.
    - Use Westsluis for shipping, taking into account the available salt balance. A possibility to limit the salt load is to restrict use around high water.
    - Blocking lockage

Based on the actual salinity in the Western Scheldt and the Canal and on the projected salinity at the end of the summer of the current year, the projected triennial average salinity can be calculated. From this, the available salt balance for the current summer can be calculated. This available salt balance for the current summer can then be translated, on a weekly basis, to an admissible number of kg of salt / day (salt load). This permitted amount of salt per day can be adjusted weekly on the basis of the available data for the salinity and the increasingly accurate predictions for the available freshwater discharges. If significant changes occur in the boundary conditions (e.g. sudden dry period, or sudden wet period), the permitted salt load may be revised after 48 hours.

The salt load per day can in turn be filled in with a certain number of lockages whereby for each of the locks in the Terneuzen complex a salt load is determined. The absolute salt load per lockage per lock will be determined as a fixed value, based on modelling. These values will be further refined after a certain period on the basis of monitoring and advancing insight.

2.2 Technical preparation group (2016-2018): practical implementation

Following the suggestion of the expert group, that a DSS could be a valuable tool, further steps were taken towards the realization of such an advisory tool, which will support the choice of the operators on the use of the locks in Terneuzen under a given range of ships and hydrological boundary conditions (upstream discharge and water height in the Western Scheldt and the Canal).

As with the expert group, the technical group consisted of specialists in shipping and water management for both countries, and representatives for the management of the lock complexes. The technical preparation group looked further into the technical requirements to implement the guidelines formulated by the expert group in a decision support system.

2.2.1 Data streams

As a first step, the necessary data streams and their respective frequency updates were identified. The DSS needs information on the hydrodynamic boundary conditions (water levels, predicted and expected discharges), water quality information (mainly for salinity) and shipping information. This information is provided by operational models and forecasts, in-situ measurements and existing tools for planning of the lockages.

Operational models and input from the lock operators upstream

The prediction values for freshwater discharges and water levels in the Canal for the Flemish section of the Canal are provided through a link with the HIC system. Upstream regulations of weirs will be assessed by the Flemish inland navigation authority to deliver a more definitive short term prediction for the freshwater discharge. This (with expert judgment) enriched information will be transmitted to the DSS by the River Information Service operators in Evergem.

The Hydrological Information Centre (HIC) is an operational service of the Flanders Hydraulics Laboratory which is part of the department of Mobility and Public Works of the Flemish administration. The HIC is responsible for measuring and predicting water levels and discharges on the navigable waterways in Flanders. It has an extensive monitoring network for this. These measurements are recorded via a telemetry system in the HIC databases. For all navigable waterways with a water management problem (with a focus on flooding) the HIC has also developed prediction models (computer models based on Mike11) with which water levels and discharges are predicted. This information (both measurements and predictions) is used to inform and warn the Flemish waterway authorities in the event of imminent flooding. This system is currently a combination of two types of models (Mike11 and Waqua) that operate within different operating shells (Floodwatch, Nautboom / Simona). The Floodwatch system is replaced by a FEWS system. It is from this system that predictions (via FTP or web services) will be made available to the DSS.

Information is exchanged every six hours with the HIC system; information on the discharges is exchanged every three hours with the operators of the upstream lock.

The communication with HIC is a two way communication. The DSS also returns data to the HIC system: the use of the locks is fed back into the operational models of the HIC system so that the predictions can be updated.

In-situ measurements and predictions

The measurement data and forecasts for the Dutch section of the Ghent-Terneuzen Canal will be provided through a link with the LMW (Landelijke Meetnet Water). The National Water Monitoring Network (LMW) is a facility of the Dutch central government that is responsible for the collection, storage and distribution of water management data. These are hydrological data such as water height, flow rate, water temperature and conductivity, and meteorological data such as wind speed and wind direction.For the DSS, the most important data from LMW are the water levels in Western Scheldt.

The communication is one-way: there is no feedback from the DSS to LMW.

Shipping information

The GTi-Tool was developed in 2012 in cooperation and coordination between the port and waterway managers, pilots, towage services and the business community. The tool is currently used for the lock planning of seagoing vessels in the locks of Terneuzen, but an extension of the tool for inland navigation is being developed at the moment. This extension will initially be used during the construction of the New Lock, to minimize hindrance to shipping. After completion of this development, the tool will be called Gti+. The GTi-Tool works on a time resolution of minutes. A lockage takes usually less than an hour and depends on the lock and the properties of the ships.

Seagoing vessels receive a confirmed time for lockage at least six hours in advance, for inland ships this is 3 hours. This means that, for inland shipping, the lock operations will be fixed from the current time to 3 hours in the future and for shipping up to 6 hours. Over a time horizon greater than the next 3 (inland shipping) resp. 6 (seagoing) hours the lock planning is still flexible.

The communication with GTI is two–way: GTI provides the DSS, in 15 minute intervals, with the information on the planned seagoing and inland ships. The DSS provides GTI with information on the time windows where locks are not available, so that the ships can be planned around these time windows.

2.2.1 Development options

For the implementation of the DSS the following options were worked out, from relatively simple (1) to very complete (3) (Deltares, 2017):

- (1) A hydraulic model, where the choices the operators make for lockage are checked against the operational preconditions. This should be considered a feedback module, rather than a decision support module
- (2) An advisory module that performs optimization on hydraulics and water quality (advice discharge, lockage) under a certain distribution of ships in the lock complex.
- (3) An advisory module that maximally optimizes both hydraulics and shipping (advice discharge, lockage and distribution of ships over the locks).

For further development, option 2 was chosen. Not only because the cost for option 3 was considerably higher, but also because the expertise of the operators have with regard to the optimal distribution of the ships should be used maximally and that this expertise is very difficult to capture in an algorithm.

Option 2 consists of a combination of an optimization tool and a hydraulic model. The combination of optimization algorithm and a matching hydraulic model is suitable to give advice on the use of the lock chambers. The planning for lockages is taken from the GTi tool and is therefore not part of the optimization.

The hydraulic boundary conditions are upstream freshwater flow and the water level in the Western Scheldt. Boundary conditions for the optimization are the operational objectives and limit values for the hydraulics and shipping planning from the GTiTool. The result of the advisory module is a proposal for lock use that meets the operational objectives and hydraulic limit values, if this is physically possible. The user can indicate a prioritization for the different operational purposes and limit values.

The hydraulic model is executed several times in succession (approx. 30 to 100 times), for this reason a simplified model is proposed; to further increase the calculation speed of the advisory module, a commercial solver will be used.

3. THE DECISION SUPPORT SYSTEM : FEATURES AND APPLICATIONS

The first goal of the DSS is to provide the lock operator with an awareness of the current scenario, whether that be high water, low water, high salinity or regular, and then to provide him with a standardized, uniform overview of all relevant parameters for that scenario : discharge quantities, water levels, forecasted discharges, shipping traffic,…etc.

In addition to raising awareness the DSS will provide the operator with a proposal for the use of the lock complex over different time scales; and the DSS will, when requested, provide the operator with all the underlying relevant information. The final decision on how to operate the lock complex lies with the operator; the DSS provides support in applying the management rules and ensures that the necessary information is provided uniformly and objectively.

Operation of the DSS will be done according to the following step, see also the schematic representation in Figure 7.

1. Calculation of the advice of DSS under the given boundary conditions and the demands for lockage. If no conflicts emerge and all requests can be met, the locks can be used for shipping and water management as planned.
2. If adjustments to the use of the locks are necessary, the advisory module initially provides advice on the use of the locks in a fixed sequence, the so-called standard solution. The standard sequence for the deployment of locks for high discharges, for example, Westsluis, Oostsluis, New Lock. Other orders apply to other situations.
3. If the standard solution is not satisfactory, the fixed sequence is released and the operator can decide which lock is to be used for which purpose (shipping, discharges, etc).For this, the DDS will provide the necessary information (for example discharge targets, (actual) discharge capacities per lock, salt loads per lockage, planned ships).
4. The DSS calculates the operator's proposal based on the current hydraulic data available in the DSS and shows the results to the operator.

5. The operator finalizes his choice and passes on the necessary lock management commands via the existing paths. The other relevant data (lock use, discharge volume) are transferred to other systems (HIC system, GTI tool).

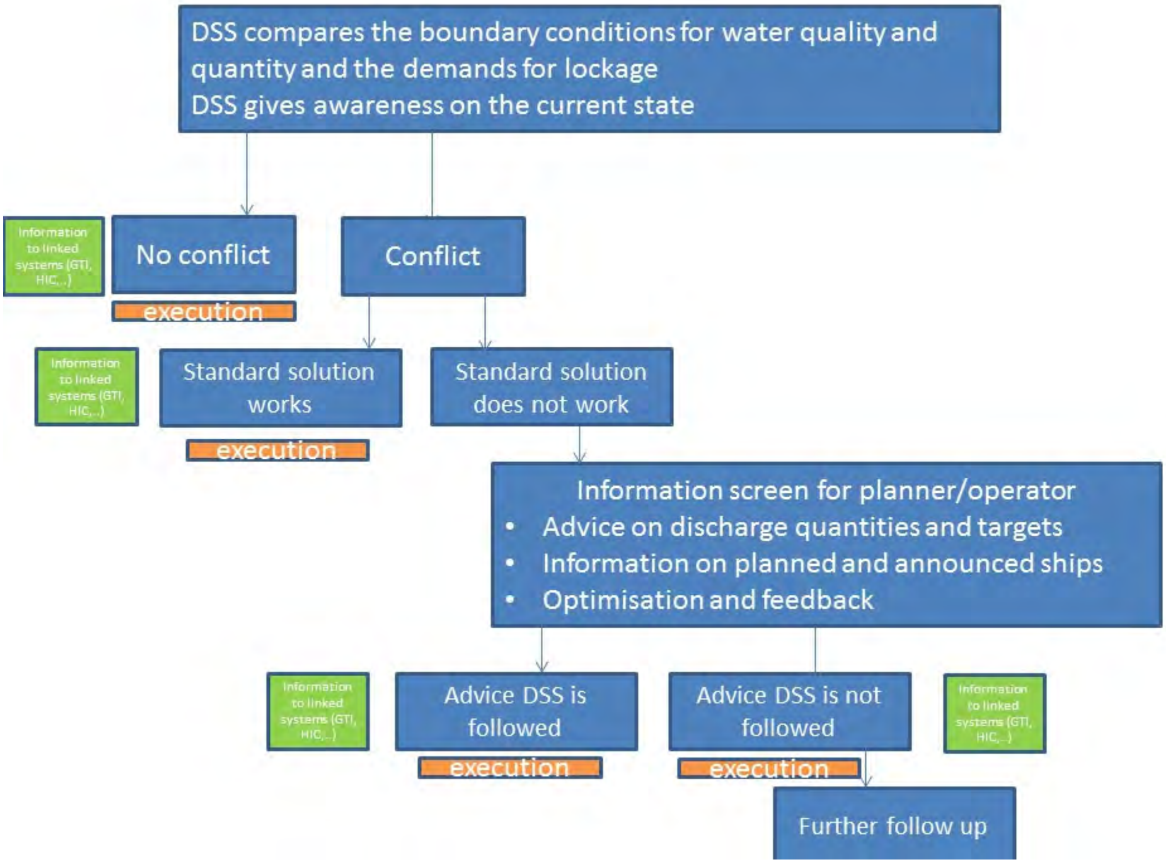

Figure 7 : schematic representation of DDS flow chart

The DSS gives also options for reporting and evaluation, which gives possibilities for optimisation of lock and weir-management up- and downstream the canal.

4. NEXT STEPS

The development of the DSS is expected to start 2nd half of 2018 in order to possess an operational system when the New Lock is taken into use in 2022. Testing of the DSS is expected to take place during the construction of the New Lock

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DESIGN GUIDELINES FOR INLAND WATERWAY DIMENSIONS

by

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ABSTRACT

The PIANC INCOM WG 141 was founded in 2010 to provide planners of inland waterways with design standards for inland waterways. The report with the title “Design Guidelines for of Inland Waterway Dimensions” will be published in 2018. In 18 meetings and three interim meetings on special questions, the group has undertaken a comprehensive view on guidelines and practice examples as well as methods for detailed design. International standards as well as practice examples show a wide scatter of recommended waterway dimensions. One reason for the differences is the great variety in traffic density but also the tradition of shipping in different countries. Furthermore, especially waterways with significant flow velocities as rivers are a complex system influenced by its varying bathymetry and currents to mention just a few aspects. So it is not appropriate to give just “one” design waterway dimension. Instead a special design method was developed, basing generally on the application of three design methods: “Concept Design Method” (coming from concept*ual*), “Practice Approach” and “Detailed Design”. Special recommendations will be provided for designing fairways in canals and rivers, bridge opening widths, lock approach length’s and widths and the dimensions of turning basins, junctions and berthing places.

The “Concept Design Method” provides basic dimensions for designing the necessary waterway dimensions. The data come mostly from existing guidelines. In a next step, called "Extended Concept Design Method", special aspects as wind or currents will be accounted for by providing formulae, derived from approximations of the driving dynamics of inland vessels. The “Practice Approach” collects and interprets data from existing waterways. It is mostly used for comparing and evaluating the results of the other design methods. If the design problem considered cannot be solved with the Concept Design Method, a Detailed Design will be recommended. It is generally basing on simulation techniques as Ship Handling Simulators. Both Concept and Detailed Design will be supported by a new approach to account for the safety and ease of navigation demands on waterway design (shortly S&E). The report provides also Guide Notes on the optimal use of ship handling simulators for waterway design. This paper provides a brief introduction into the structure of the report of WG 141. It outlines the main findings, especially concerning the consideration of the necessary S&E quality. Selected results will be presented concerning the Concept Design Method of fairways in canals, the Practice Approach for rivers and the Detailed Design for bridge openings.

1. INTRODUCTION

1.1 Motivation to install WG 141

One of the motives for founding PIANC-INCOM WG 141 “Design Guidelines for Inland Waterways” was the lack of internationally accepted guidelines for inland waterway dimensions, in contrast to regulations for sea-going ships. Another reason to update existing knowledge of waterway design corresponds to the change in fleet, especially with an increasing part of longer, wider, deeper going and stronger powered vessels and consequently the dimensions of the design vessels. These new vessels are generally the reason why wider lock chambers, lock approaches and fairways are needed.

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On the contrary, these new vessels are generally better equipped than traditional vessels, e.g. with two thrusters instead of one, with twin rudders instead of single ones or with bow thrusters and passive bow rudders in some cases. This development, combined with a general reduction of the number of ships sailing on inland waterways and better information services, which are available now and will be more capable in future, provides an opportunity to potentially restrict the lateral dimensions of the navigation channels despite the larger widths of the vessels.

1.3 Differences to sea-going Vessels

Furthermore, in contrast to sea-going ships, the traffic with inland vessels is generally less dangerous, for example collisions with bank protections are more or less a normal situation when travelling in inland canals. One reason is that sea-going ships are less powered and worse steerable related to their dead weight and drive with comparatively high ship speeds, forcing high safety standards. Thus, design standards for sea-going ships as those of PIANC MARCOM WG 49, have to be more spacious for safety reasons and are thus generally not applicable to inland-going ships. But, of course, the safety and ease of inland navigation must be ensured nevertheless and restricts the possibilities of “narrow solutions”, which will be demanded for e.g. from ecological and politico-economic reasons.

1.4 Minimum Waterway Dimensions

Therefore, there is a need to specify the minimum necessary requirements on waterway dimensions, especially from the nautical point of view. This does not mean that WG 141 proposes these minimum dimensions. In contrary, looking on the aspects of safety and ease of navigation (in the following shortly “S&E”) and the operational economy of shipping, the design should be generally as generous as possible taking into account for example a possibly changed traffic density in future, but, looking especially on impacts on the environment, socio-economic aspects or the politico-economics of the waterway improvement, the design should be as narrow as necessary - but not more than that. So, it makes sense to define just these lower limits to avoid needless discussions with opponents of waterway improvement measures. This is the main task of PIANC-INCOM WG 141.

2. OBJECTIVES OF THE REPORT AND GENERAL REMARKS ON THE APPROACH

2.1 Target Group

The report of WG 141 aims mainly on planners of inland waterways and the infrastructure. The planners will be supported

(1) By providing concrete numbers on nautically necessary minimum waterway dimensions for selected types of infrastructure (results of the so-called "Concept Design Method" and "Practice Approach" and
(2) Process recommendations for performing a detailed study for specifying necessary minimum waterway dimensions, using field data, scale model test or in most cases simulation techniques as Full Bridge Ship Handling Simulators.

Decision-makers are the 2nd important part of the target group. They will benefit mainly from

(1) The provision of generally applicable design rules as the procedure to perform the design in a structured and objectified way ("General Approach in Waterway Design", Figure 3) or
(2) Information on the necessary effort to follow this procedure, especially on appropriate data, hydrodynamic and nautical models as well as number of simulations runs to obtain results in the desired quality.

The report supports also experts developing and applying simulators, especially concerning the proposed optimal way to perform a detailed nautical study up to all the possible stakeholders involved in a waterway design project (including layman) who are interested in all the problems or special features related to e.g. he driving dynamics of inland vessels and its impact on waterway infrastructure design. So, the report offers information for a large target group concerning nautical aspects of design.

2.2 Main Objectives

The central problem of the working group process was to consider the partly huge differences in recommended waterway dimension in relevant international guidelines (results of the Concept Design Method). These differences can be interpreted as to be mainly caused by varying necessary safety and ease of navigation demands and corresponding traditions of shipping in these countries. This problem was overcome by the objectification of different necessary ease qualities in waterway design. The corresponding design approach will be called "Safety and Ease of Navigation Approach" or shortly "S&E" in the following. Correctly applied, it matches the different recommendations to a large extent. Hence, the main and primary objective of the report is to ensure that the results of the Concept Design Method proposed in the report are rationally understandable and comparable to the application of different national guidelines and its different S&E demands.

In addition, the design of inland waterway infrastructure according to its main dimensions should follow a structured, standardized procedure, considering all the necessary information e.g. on relevant local boundary conditions and the quality of methods to be used. This is, to ensure that the most suitable design approach including corresponding methods will be used and not e.g. the by accident available methods. Finally, the report promotes understanding of required funds and efforts to perform the design approach in the recommended way, especially if simulations will be used. This concerns e.g.

- the necessary data base,
- the type and capabilities of models,
- the need to calibrate and verify the models and
- the way to apply models in a comparative way as it is usual in applying hydraulic models but not commonly accepted standard in using Ship Handling Simulators (“SHS” shortly in the following),

up to the way how model results can be interpreted in an optimal way to eliminate modelling inaccuracies to a large extent. This, because the efforts and funds for designing waterway infrastructure measures are generally higher compared to the use of simulators for pilot training. On the other hand, the expenses for a comprehensive nautical study are generally only a small fraction of the construction costs and seem thus to be justified from the viewpoint of experts involved in WG 141.

2.3 Waterway Dimensions considered

The report focuses on waterway infrastructure measures that cause generally the highest construction and compensatory expenses (economic view) as well as the highest ecological footprint. These are

- the width and depth of canals,
- the width and course of fairways in rivers,
- the width and height of bridge openings,
- the length and width of lock approaches (not the harbor or the lock),
- the navigational space at junctions,
- the dimensions of turning basins and
- the length, width and layback of berthing areas.

The report focuses on nautically design aspects and minimum dimensions only. These limitations were necessary in order not to exceed the framework of the report, but economic and ecologic demands will be considered fulfilled in this way too.

2.4 Application Limits of the Report

The report gives no answer, *whether* a waterway infrastructure project shall be realized or not. This fundamental decision is part of an intensive justification procedure, including the politico-economical appraisal and the tolerability concerning water management, land use and ecology, that must be carried out before the nautical design can start. This part of the design process is illustrated in the following sketch, Figure 1. But the report provides answers on *how* the infrastructure should be realized, if the fundamental decision was made – and especially considering the necessary demands of S&E. Note also that, the report provides answers to design questions only up to a certain degree of concretization, not more, but also not less. This restriction is necessary because of the generally highly complexity of each waterway design problem with all its different local boundary conditions and constraints.

Therefore, the WG 141 report provides in many design cases no specific numbers, e.g. of the necessary width of a lock approach, but “process considerations” on *how* to perform the design in a structured manner. Therefore, the report doesn’t replace detailed engineering works. But it explains in detail the proposed approach how to obtain appropriate waterway dimensions, e.g. by considering and filtering all nautically relevant design aspects and giving hints on data and models needed and how they can be used in an optimal way. This contribution of the guidelines to the planning process is illustrated in Figure 1.

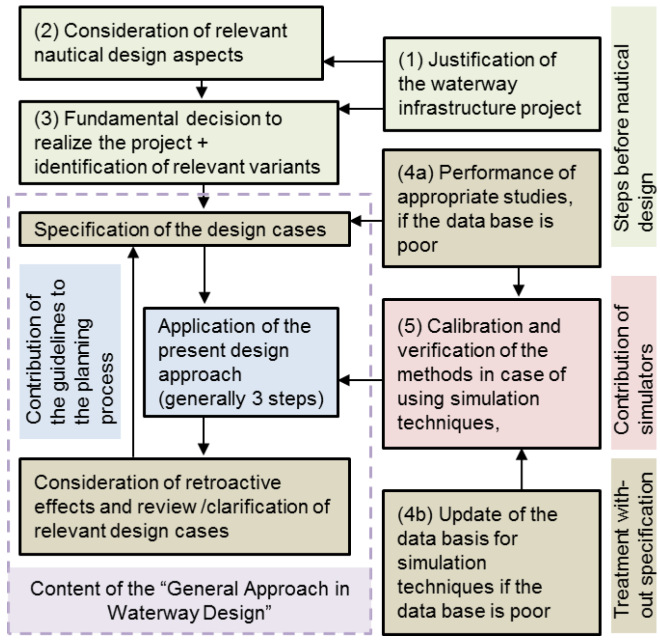

Figure 1: Contribution of the WG 141 report to the planning process of a waterway infrastructure project

2.5 Types of Recommendations

Nevertheless, there are several design cases, allowing to specify selected waterway dimensions by considering existing guidelines only, that is, without looking additionally on practice data or without performing a detailed study. This is e.g. the case for the necessary widths and depths of canals, if the necessary S&E demands can be chosen very accurately. The corresponding design approach (1st type of recommendation) is rated to be the most concrete and precise one, see Table 1, illustrating this ranking in comparison to the other types of recommendations, explained in the following.

By contrast, the design of waterway dimensions, where the local boundary conditions as the course of the river, the bathymetry and the flow velocities in a lock approach area are different from case to case, demand for a Detailed Study and thus for process recommendations only. Although these are within themselves very specific, e.g. concerning the check of the applicability of the models used, they may be ranked according to its degree of concretization to be very low (type 3, less concrete).

Between these poles, recommendations are derived from practice data (type 3 in table 1). They may be very concrete as the fairway widths in rivers, but the scatter of the available data is very large and thus the recommendations are less accurate compared to data from canals. Because of this fact and because local boundary conditions as the course of the river and its flow distribution may vary from case to case, these recommendations demand generally for further detailed considerations.

A 4th type of recommendation to choose appropriate waterway dimensions may be used as a first step in applying simulation techniques. The reason for this step is that e.g. the hydraulic model, specifying inter alia the flow velocities in the design, but also the visual model in a SHS needs the to be specified and corresponding hydraulic calculations have to be made before the actual simulation can start. These recommendations are of course less specific and inaccurate, but nevertheless necessary to start the design process.

Type	Description	Example	Degree of concretization and rating
1	Recommendation derived from applicable existing guidelines	Concept Design Method of canal dimensions	Highly concrete and accurate
2	Recommendation derived from practice data	Concept Design Method of fairways in Rivers	Concrete, but less accurate compared to canals - requires generally a Detailed Design
3	Process recommendations for performing the Detailed Design	Usage of simulators for lock approaches	Highly concrete concerning the approach, but no numbers
4	Preliminary Design - waterway dimension derived from driving dynamics - 1st step of the Detailed Design	Lock approach width accounting for cross currents to specify bathymetry and flow field for subsequent simulations	Concrete, but less accurate because only a few influencing parameters considered

Table 1: Types of recommendations and rating according to the degree of concretization and precision for waterway design

3. REVIEW OF THE WG PROCESS

The WG was founded in the year 2007 within the framework of the PIANC World Congress in Liverpool. After 18 meetings, 3 interim meetings, 4 participations to PIANC conferences with 2 workshops and 16 publications, the group prepared the last, from INCOM members reviewed version end of March 2018. The publication is expected in summer 2018. The reason for the long-lasting working process is the complexity of the design problems, related to the various boundary conditions in inland navigation. A special problem was the way how to account for the huge number of influencing parameters, which led to the S&E approach.

The working group consists of experts from China, Europe and the USA. Involved were by profession civil engineers, naval engineers and a former captain. They are employed at waterway administrations, engineering bureaus, institutes of applied science and developers and users of ship handling simulators.

Starting with the discussion and specification of the Terms of Reference, given by INCOM, the group dealt with the following points:

- Viewing, preparation and evaluation of existing international waterway guidelines according to minimum waterway dimensions and corresponding safety and ease quality.
- Development of an experience-based design from the interpretation of existing guidelines, called Concept Design Method, and assignation of corresponding S&E qualities.
- Definition of S&E categories and development of an assessment scheme for necessary minimum S&E standards with a scoring system to support the Concept Design Method, called “Simplified S&E Approach”.
- Evaluation of guidelines, relevant literature and approaches, basing on the driving dynamics of inland vessels with regard to extra widths (generally called increments”) as wind surcharges.
- Collection and analysis of existing waterway infrastructure, leading to the Practice Approach.
- Evaluation of criteria concerning the application limits of the Concept Design Method, together with European developers and appliers of SHSs (BAW, DST, Flanders Hydraulics, MARIN, SIPORT21).
- Development of an approach to optimize the application of SHSs for waterway design purposes, called “Detailed S&E Approach” and coordination with the aforementioned appliers of SHSs.

A testable draft of the report was worked out in the end of 2016, leading to first comments by INCOM. A second draft was worked out to be checked by a reviewer group in summer 2017. Basing on these comments, a second last version was worked out up to January 2018 and was the basis of final comments from INCOM members. The version, which is discussed here, is dated end of March 2018. It is the basis for the expected final approval from PIANC.

4. STRUCTURE OF THE REPORT

Before going into details of the report, whereby the content of each important chapter will be presented briefly, the headings of the major chapters are given as follows:

1 INTRODUCTION
2 TECHNICAL INFORMATION
3 APPROPRIATE ASSESSMENT OF SAFETY AND EASE QUALITY AND ITS USAGE FOR DESIGN
4 RECOMMENDED METHODS FOR WATERWAY DESIGN
5 RECOMMENDATIONS FOR SELECTED DESIGN ASPECTS
6 CONCLUSIONS
REFERENCES
GLOSSARY
APPENDIX A: SUMMARY ON EXISTING GUIDELINES
APPENDIX B: DIMENSIONS OF EXISTING WATERWAYS - PRACTICE
APPENDIX C: APPROPRIATE ASSESSMENT OF SAFETY AND EASE QUALITY AND ITS USAGE FOR DESIGN
APPENDIX D: DETAILED OR CASE BY CASE DESIGN – USING SIMULATION TECHNIQUES OR FIELD INVESTIGATIONS
APPENDIX E: EXTENDED CONCEPT DESIGN METHOD – ACCOUNT FOR EXTRA WIDTHS
APPENDIX F: APPLICATION OF THE DETAILED DESIGN APPROACH TO AN EXAMPLE

It should be noted that the report is written in a way to be read selectively. So, experts, e.g. planners of waterways, may extract their specific information only from Chapter 5. It collects the results of the Concept Design Method and Practice and provides hints for Detailed Design for selected waterway dimensions. Maybe experts will have a look into the appendixes too, which go into further detail, e.g. how use the Extended Concept Design Method (APPENDIX E) or the recommended approach for optimal usage of SHSs (APPENDIX D). Decision-makers may be interested also in Chapter 4, which shows the recommended design procedure in detail, especially concerning the main questions to be answered and who should be involved in waterway design. Readers, who demand for understanding the recommended approaches, may have a special look into chapters 1 - 3 and 6, as well as appendix E. They will find comprehensive information about the relation between minimum waterway dimensions and the driving dynamics of inland vessels. Hundreds of links between the chapters facilitate the selective reading.

5. EXAMPLES FROM THE CONTENT OF THE REPORT

5.1 Chapter 1, Introduction

The main content of Chapter 1 was already reflected in this paper in the chapters 1.1, 1.3, 2.1, 2.2 and 2.4. The main issues are be outlined again in the following:

- Because of the strong demand to consider hydraulic engineering, economic and ecologic aspects all together and the general request from waterway planners for narrowly standards, the report focuses on minimum waterway dimensions and from the nautical point of view only, because the other aspects will be accounted for to a high extent automatically. This does not mean that WG 141 recommends these minimum standards. By contrast, the waterway design should generally be as generous as possible to facilitate shipping. But there are often unavoidable strong constraints demanding for minimum standards, but they should not be smaller than those proposed in this report to ensure the safety and ease of navigation.
- Because of the numerous influencing parameters and the complexity of inland waterway design aspects in general, a “Three Methods Approach” was proposed, basing on the traditional Concept Design Method, extended by the Practice Approach and Detailed Design where appropriate. So, not only specific numbers will be provided in the report as in existing guidelines, but also process recommendations how to proceed in an optimal way, especially concerning the use of ship handling simulators if application limits of the Concept Design Method are exceeded.
- Because of the large target group of the report, it was written to allow for selective reading.

From the content of Chapter 1, it should be additionally mentioned that,

- It was relevant to limit the review of existing guidelines and practice examples to commercial navigation, since large vessels scale generally the waterway infrastructure dimensions.
- The restriction to minimum waterway dimensions accounts for the very important issues of Climate Change, e.g. mitigation measures for flood and drought to a large extent automatically too and thus, did not have to be considered separately.
- Special attention will be paid to the MARCOM approach for Harbour Approach Channels (PIANC, 2014). It was used as a basis for the proposed Concept and Detailed Design for inland navigation, but for special design cases only and – of course – with somewhat other figures concerning the necessary minimum waterway dimensions.

5.2 Chapter 2, Technical Information

This chapter is mainly written to understand the relationship between the design of waterway dimensions and the special properties and features of waterways, its infrastructure and the properties of vessels and its driving dynamics. The chapter starts by looking on vessel types, the typical features of free flowing rivers, over impounded rivers up to canals, explained by examples as the various construction details of locks in rivers. One example shows the following details. One extreme example concerning a strongly reduced safety and ease of navigation is shown in Figure 2. It's a lock without an upper or lower harbor, that is, without a quiet-water zone, but with a "gliding pear" to facilitate the intake into the lock. Such a solution will – of course – not be recommended by WG 141, but it shows the variety of existing waterway infrastructure. To match this variety of possible solutions was the main concern to be solved in the report.

Figure 2: Multiple locking of a pushed convoy in the USA

The chapter then provides comprehensive information about the physics behind the vessel behavior, e.g. the strong increase in resistance in shallow and sideways confined waters as canals. This increase has consequences e.g. concerning the optimal way to be considered in simulations. If e.g. formulae will be used made for shallow water, but not taking into account the sideways constraints, the rudder forces at the same vessel speed may be underrated and thus the navigability of the modelled vessel. Finally, information about ship-induced waves and currents will be given to understand the various constraints of vessels driving close to banks, to other vessels or close to infrastructure elements as spur dykes, which lead to accordingly safety distances.

Chapter 2 provides also check lists of boundary conditions to find out and specify relevant design cases. According to Figure 4, this is one of the most important steps in waterway design, because there is generally a huge number of influencing parameters as water levels, flow velocities, environmental conditions like wind and fog, ship types and loading, traffic situations and so on. Considering the probability of occurrence, criteria will be given to extract possibly decisive design cases in order to reduce the design expenses, because the design approach recommended herein with different steps should then be applied to all these cases. Thus, Chapter 2 is written not for experts, who are of course familiar with this information, but for laymen or decision makers, who want to know more about the special problems of waterway infrastructure design and to assess the possible expenses of a nautical study.

5.3 Chapter 3, Appropriate Assessment of Safety and Ease Quality and its usage for Design

As stated earlier in chapter 2.2 of this paper, there are partly huge differences in recommended waterway dimensions in different national guidelines (Concept Design Method), e.g. lock approach lengths, which reach from about 4 times the vessel length L for some waterway classes in China down to 0.5·L in France – and practice data show even lower values as demonstrated in Figure 2. But there are of course objective reasons for these differences and the different S&E qualities may be associated with it. Therefore, the problems to be solved by WG 141 in order to end up with an <u>appropriate consideration of S&E for applying the Concept Design Method, called “Simplified S&E Approach”</u> were,

1. to explain these differences, e.g. by looking on the traditions of shipping in these countries,
2. to learn from the different guidelines to find out *appropriate* S&E qualities and
3. to cope with the generally huge number of design criteria and influencing parameters.

Some of these parameters are shown in the following Figure 3. They determine the existing (analysis case) or the necessary (design case) S&E quality of a driving situation considered. They are collected into waterway-related parameters (blue boxes, containing fairway and environmental conditions), vessel- and speed-related aspects (red boxes, collecting loading, vessel types and instrumentations as well as information systems) and traffic/human-factor related design criteria. You can find these parameter groups in the final table of the S&E approach, see here Table 4. Note that all these parameters don’t act in the same direction. E.g. challenging fairway condition speak for a higher necessary ease category in design, whereas a lower ease quality may be acceptable for design, if good on-board information systems are available. Therefore the usual way to cope such influences as in the MARCOM Approach for sea-going vessels, by adding increments to basic waterway dimensions in case if the driving conditions get more complicated, seems not to work optimally in our cases.

The evaluation of these influencing parameters forms the backbone of the Simplified S&E Approach to be used for the Concept Design Method. It leads in the end to an S&E score and thus, to a specific S&E category. If this S&E approach works properly, it should explain the differences in the various guidelines and thus fit with these guidelines to a large extent. This was the main objective to develop the approach – and, each member of the WG must be able to recognise himself in the report!

The solution of the aforementioned problems lead to,

- the definition of S&E categories (qualities), three in total for simplification, named:
    - Category A: Nearly unrestricted drive (as on the Lower Rhine River),
    - Category B: Moderate to strongly restricted drive (as the Upper Rhine, The Mississippi, Dutch “normal” Canals etc.)and
    - Category C: Strongly restricted drive (as Dutch “narrow” Canals and German Canal cross sections, narrow bridge openings, sailing at lock approaches etc.),
    - whereby the *safety* should be ensured also in the lowest category so that extreme conditions as low speed manoeuvring situations while e.g. entering a lock chamber were not considered as well as very comfortable conditions as one-lane traffic in a channel designed for two-way,
- the allocation of recommended waterway dimensions from different guidelines and practice to the aforementioned three categories, see application example in Chapter 6 of this paper,
- the assessment of the S&E category by
    - using a catalogue of different criteria, reflecting the influencing parameters of Figure 3 (see Figure 10), and
    - creating and adjusting a scoring system to end up with a comprehensive “Simplified S&E Score”, which will be assigned to the three ease categories,
- the calibration of the scores by checking, whether the evaluated ease categories fits with the expected ease quality from experience and the assessment of the WG members and, finally,
- the linking between the categories and appropriate waterway dimensions by comparing with existing guidelines and practice.

In the end, the Simplified S&E Approach objectifies (by different criteria) and quantifies (by the score and the assignation of score to ease category) the demands of safety and ease of navigation for applying the Concept Design Method. The application will be shown in Chapter 6 with Figure 10 of this paper by an example.

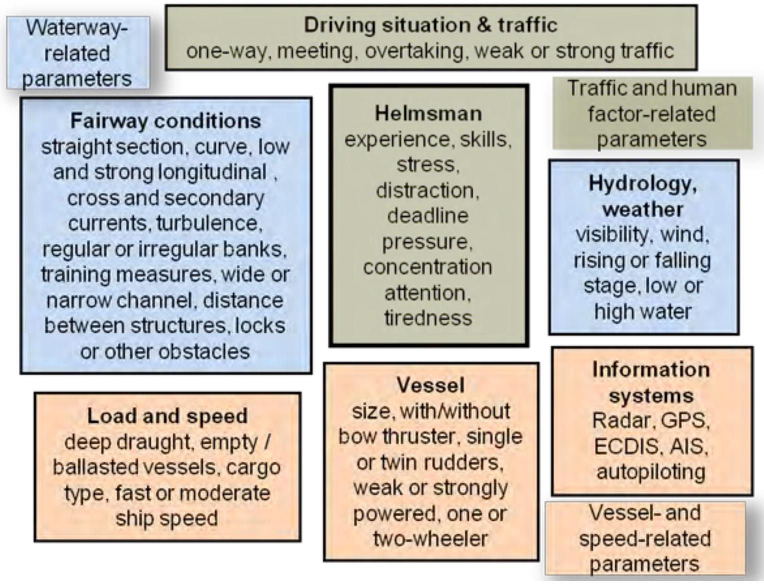

Figure 3: Influencing parameters in waterway design

The Simplified S&E Approach replaces the application of existing national guidelines in cases where they are not applicable or for countries having no own guidelines according to the main objectives of PIANC reports. The approach replaces also a risk based design which is commonly used nowadays especially in maritime waterway design. If there are e.g. buildings, quay walls, floating facilities or vessel berths in the vicinity of the navigational area compared to a sloped bank, the necessary S&E category will be higher and thus the necessary fairway evaluated with the Concept Design Method. So, safety allowances will be met indirectly by demanding for a higher ease quality.

A similar way to objectify and quantify the S&E demands will be recommended in the WG 141 report also for the Detailed Design (performing a detailed study), especially if it is performed by using simulation techniques or field investigations. The corresponding approach will be called “Detailed S&E Approach”. It bases on the following principles:

- The S&E quality of a driving situation, either observed in reality or in virtual reality, will be quantified by evaluating time series of relevant (simulated or measured) parameters
  - as rudder angles, scaling the effort to control the driving situation and
  - e.g. ship-bank distances, which are associated with the exploitation of existing resources or the deviation from a desired way of driving,
  - using a scoring system to allocate e.g. the difficulty of a driving system to a score, to average the time-series of these scores in design-relevant waterway stretches
  - and matching the score-averages of different parameters to one comprehensive score, named “Detailed S&E Score” analogous to the Simplified S&E Score.
- For assessing the S&E of the driving situation considered, the latter will generally not be evaluated directly, but by comparing it to a reference case (principle of comparative variant analyses), see also Figure 6 (dashed frame collects the methods used to perform the Detailed S&E Approach). By this means, unavoidable inaccuracies of the simulations will be eliminated to a large extent. So, always just two variants will be considered, e.g. the design case and an appropriate “ease reference case”, whereby
  - the ease reference case may be chosen by applying the Simplified S&E Approach first to the design case, in order to assess the striven S&E quality, and then to different possible reference cases, to check, which reference case fits best with the Simplified S&E score of the design case (often identical to a special situation of the present nautical conditions, whose S&E standard should be reached also after design),
- and comparing the comprehensive Detailed S&E Scores of both the design and reference cases, which are evaluated from the results of e.g. the simulations with a SHS of both variants, using the aforementioned procedure to quantify the S&E.

- Also the influence of e.g. the skills, the attentiveness or destruction of the helmsman, usually called “human factor”, can be quantified to make it comparable between variants. The report proposes for this purpose the NASA TLX-Test (Task Load Index).

But not only these quantified S&E scores of the design and reference case should be used for assessing the acceptance of the design (relative results), of course also the simulated results *directly*, even if the inaccuracies are larger than by applying the principle of comparative analyses, e.g. the minimum bank distances (absolute results), together with the comments of the pilots and local experts facing the results (“weak design criteria”) are very important. Thus, the last and most important step in performing a detailed study is the proper *interpretation* of all available results, where appropriate also in comparison to previous projects, but of course with special respect to the application of the Detailed S&E Approach.

5.4 Chapter 4, Recommended Methods for Waterway Design

Because of the large number of influencing parameters as visualized in Figure 3, the different parties and stakeholders involved, the strong impact on the water sector and its ecology in general and the numerous interconnections to different investigations and studies, inland waterway design is generally not a standalone or straightforward procedure. By contrast, it is embedded in the planning concept as indicated in Figure 2 and it needs generally several adaptions during the planning process. But even the waterway design itself needs adaptions, e.g. concerning the determination and specification of decisive design cases or the feasibility of boundary conditions as the spacious consideration of wind effects, looking on local constraints or economical limits. Therefore, waterway design is generally a “looped approach” as indicated in the flow chart of Figure 4, meaning that the feedback after performing the design may lead to question the boundary condition of the design and especially the chosen decisive design cases. The WG 141 report offers several of these flow charts as shown in Figure 4 to visualize the recommended procedures with its interconnections to other design-relevant activities and aspects. One of the most important charts is presented here somewhat simplified in Figure 5. It shows how the aforementioned “Thee Methods Design Approach” works and in which cases the performance of the Detailed Design can be dispensed. Because the flow chart is almost self-explaining, only some remarks should be added here.

After specification of the special design case considered, the application of the “Three Methods Design Approach” procedure starts with the application of the Concept Design Method (upper and right part of Figure 5) It is generally the same as if existing national guidelines will be used. National Guidelines reflect the best practice in this country. Its application also fulfils the requirements of standardisation. Therefore, Countries with their own guidelines may insist to use their guidelines only and the design may end here without additionally applying the other two design methods additionally. But in the opinion of the authors of the WG 141 report, it makes sense to look even in this case to look additionally into applicable international guidelines, to consider practice examples (Practice Approach) or design results from previous projects and, – of course – to use the recommendations of WG 141 in Chapter 5 of the report, considering the S&E approach outlined above to objectify the design. If the results are about the same and if there remain no doubts about the applicability of the methods, the design may end here also after the recommendations of WG 141 (recommendation type 1 in Table 1). This is generally the case for canal design, also because simulations are not that accurate for driving situations in shallow water and additionally in strong sideways confined water because of the strong influence of ship-induced currents on the driving dynamics.

One reason for the recommendation to compare all these results is that, Guidelines will be adapted often too late to account for new developments e.g. of a changing fleet. Hence, they are sometimes backward looking and they may hinder or hold back necessary developments. And, as not all relevant design aspects will be treated in the guidelines, this fact may narrow possible innovations and hinder adapted solutions regarding locally different boundary conditions. To point it out once again, the WG 141 report expands the usual Concept Design Method by considering both national and international guidelines, together with own recommendations and combines it with the Practice Approach. This is, above all, to increase the reliability of the results.

If there are large differences between the determined numbers, especially between Concept Design and Practice, if application limits are exceeded or generally if the there are other good arguments for more detailed investigations, the next step should be carried out, which may base on field investigations, scale model tests or nautical simulations as shown on the left hand side and the bottom of Figure 5.

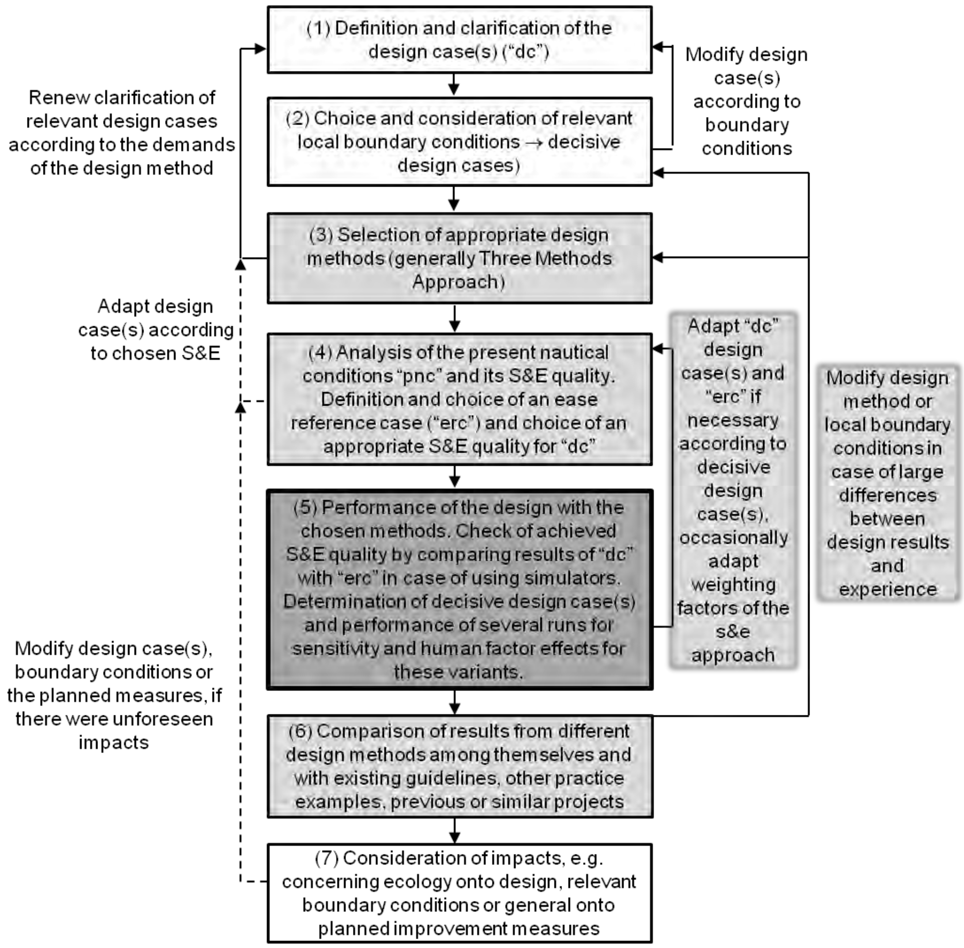

Figure 4: General Approach in Nautical Waterway Design

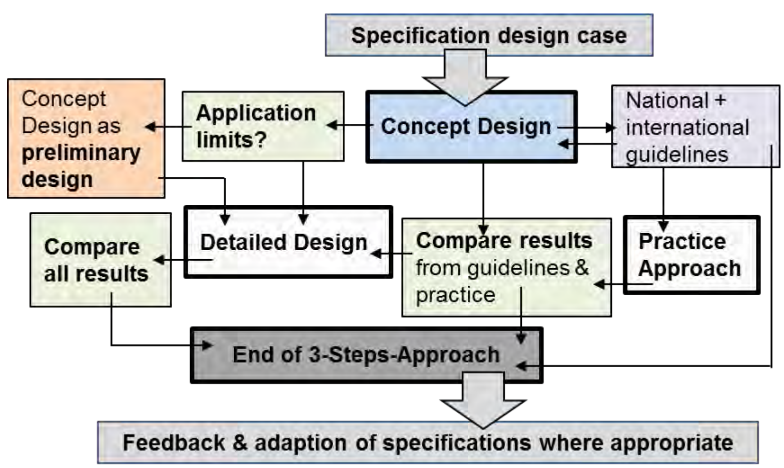

Figure 5: Overview for applying the Three Methods Approach

The results of the Concept Design, if necessary extended by increments (Extended Concept Design), may then be used as a preliminary design for the detailed study (recommendation type 4 in Table 1), because, as mentioned earlier, the latter needs the waterway dimension to be designed first before it can be checked e.g. by using e.g. simulations. This is, because e.g. the bathymetry and the flow models of the relevant waterway stretch are generally depending on the waterway dimensions to be designed itself.

The next step of the “Three Methods Approach” is to choose the methods for performing the Detailed Design, e.g. scale model tests compared to Ship Handling Simulators. For this purpose, the report provides corresponding check lists in Chapter 4. If e.g. the ship-induced wave and current system dominates the vessel behaviour as in very narrow canals, the Method of choice are scale model tests. Then the chosen method should be applied with special respect to the Safety and Ease of navigation as outlined in Chapter 5.3 of this paper. For this purpose, the report provides detailed process recommendations (Type 3 in Table 1) how to perform a detailed study in an optimal way, especially to reduce modelling inaccuracies (Figure 6). To finish the “Three Methods Approach”, the results of the Detailed Design should be compared with those from Concept Design and Practice, even if application limits of the simplified methods were exceeded - and if possible with similar available projects! Only if the differences of the results may be acceptable or explainable and if there were no doubts about the applicability of the chosen design methods, the Three Methods Approach may end here. It follows the consideration of retroactive effects on the planning process and thus the feedback to the planners as illustrated in Figure 5 below, which may lead to rethink the relevant boundary conditions and thus the design cases itself, see also Figure 4 (arc running backwards from step 7 to 1). Note at this point that the report provides particularized descriptions on the optimal way how to perform detailed studies, especially using SHS, not only in Chapter 4 of the report, but also more comprehensively in Appendix D. In this paper, only a highly simplified flow chart will be presented, Figure 6. It shows e.g. the steps which are required to account for the Detailed S&E Approach as outlined above (dashed frame).

Without going into details, Figure 6 demonstrates impressively that the performance of simulation runs in e.g. a Full Bridge Simulator, needs a lot of things have to be done first, in parallel and after the runs and in a structured procedure. Some aspects may be highlighted as follows:

- Check the data base for simulations before starting with simulations, inter alia the bathymetry, because the forces of the underwater body of a vessel strongly depend on the draught to water depth ratio, but also on the flow field. This especially concerns to the influence of water level slopes and secondary currents if relevant. Remember the saying “garbage in, garbage out” of experienced modellers in case of insufficient data.
- Calibrate (e.g. by simulating well-known driving situations, comparing results and adjusting modelling parameters where necessary) and verify (using data that are not taken for calibration) the models using appropriate, well-known driving situations! Generally, the present nautical conditions can be used for model verification, because local skippers and authorities, who should take an active part in the simulations, are very familiar with the local conditions and know how the vessels behave in reality. But please use *quantitative* information, e.g. measured swept area widths, not only the “feel” of local skippers for verification.
- Choose appropriate reference cases, e.g. for simplification a driving situation from the present nautical conditions (often the nowadays largest permitted vessel under the most critical, but “just drivable” conditions, whose S&E quality should at least be reached in design), if the driving situation is comparable to the design case (waterway structure, e.g. lock approach area, vessel type and loading and draught to water depth ratio, traffic situation as encounter or overhauling) and compare each design case with its corresponding reference case.
- “Scan” possible design cases (e.g. water levels, together with loading conditions, vessel types, traffic situations, extra forces as those from wind etc.) by simulations with less effort, e.g. without comparing them to reference cases (trust the “absolute” results in this phase if the results are clear) and perform only one run instead of several runs as for the decisive design cases. The results of the latter should be supported by several runs (needed to verify results and to account for human-related effects – “one run is no run”).
- Interpret the results properly, looking not only the Detailed S&E Approach, but also on *differences* e.g. in the bank distances between design and reference case, on *absolute results* as well as “weak” information as the “feel” of skippers and comments of local experts. As outlined here in Chapter 5.3, a proper *interpretation* of results is just as important as the proper modelling of the driving situation considered.

Addressed to the clients of detailed studies, the WG 141 authors would wish that additional expenses according to this approach will be reimbursed in order to end up with reliable waterway dimensions. Note that expenses for nautical studies are generally minor to construction costs, but may lead to neither undersized nor oversized solutions. Addressed to the users of simulators, the authors would wish that they offer these extra services to their clients and convincing them to pay the extra costs.

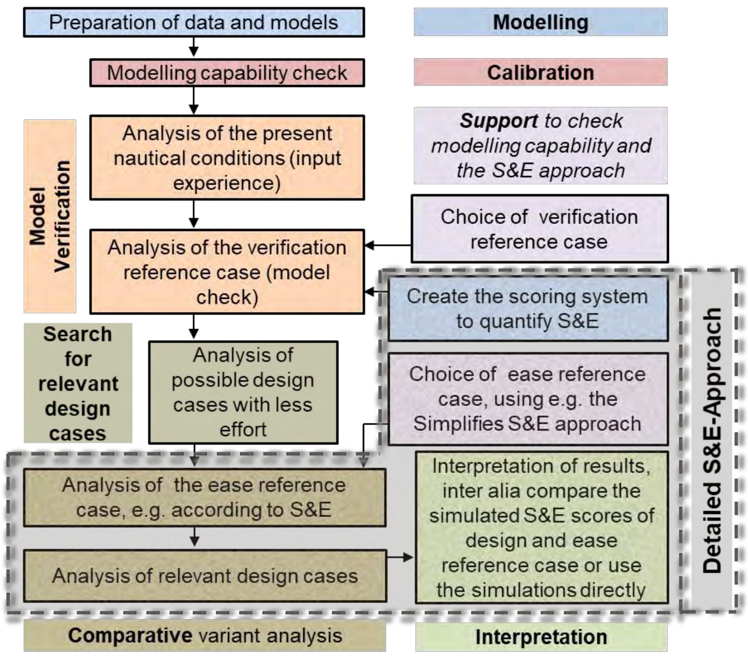

Figure 6: General recommendations and steps to follow using SHSs for waterway design with special respect to Safety and Ease of navigation (dashed frame)

The authors of WG 141 report are of course aware, that the reality is often different to this “ideal approach”, meaning that simulation will be run in many cases without proper verification of the models or without comparison to appropriate reference cases. Nevertheless, the authors hope that, if PIANC demands for an appropriate modelling procedure, the opportunity to use simulators for waterway design purposes – and we only talk about this application area of e.g. simulators, not e.g. for training of pilots – will be taken in a more sophisticated way than is customary today, so that the quality of nautical studies can be increased. Note again that waterway dimensions generally “scale” the expenses and consequence of waterway improvement measures, so that possible somewhat higher modelling expenses may be justified in any case.

5.5 Recommendations for selected design aspects

As mentioned above, the waterway dimensions of fairways in canals and rivers, bridge openings, lock approaches, turning basins, junctions and berthing places will be considered in detail in subchapters of Chapter 5.5. Each subchapter starts with the definition and clarification of variables as the coverage ratio n (cross section area, divided by vessel’s midship cross section) in case of canal design. Then existing guidelines will be analysed according to the waterway dimension considered. From this, the recommended numbers of the Concept Design Method will be extracted and put in corresponding design tables. One example shows Table 2, containing data referred to fairway dimensions of canals (fairway width W_F, water depth h). Note again, that these values are minimum figures from nautical aspects only and that they are valid for restricted boundary conditions only.

In case of Table 2, these restrictions are as follows:

- Vessels with average equipment and average instrumentation, sailing cautiously, regardful and with moderate vessel speeds (relative to water) around v = 0.5 (encounters) up to 0.7 (normal drive) times the critical speed in a straight canal section. The assumed speed reduction at manoeuvres is important to restrict interaction forces and thus corresponding safety distances, which are included in the basic widths in the design situation considered.
- The application is further restricted to the limits of minimum curvature Radii R, maximum flow velocities v_{flow}, cross flow velocities and so on as well as traffic densities up to about 30,000 cargo vessels/year. But even if the specified minimum radii, maximum cross flow and wind velocities are not exceeded, corresponding extra widths must nevertheless be accounted for, because the basic widths are valid for straight canal sections without crossflow and crosswind only. Also extra widths are recommended for higher traffic densities than specified.

Waterway	Fairway width for alternate single-lane				Fairway width for two-way (approximately also for two-lane, including overtaking manoeuvres)			
	Ease quality			Remarks	Ease quality			Remarks
	C	B	A		C	B	A	
min W_F (straight canal sections)	**2·B** *1.9·B*	*2.1·B*	*2.3·B*	For security reasons	3·B *2.8·B*	4·B *3.5·B*	*4.5·B*	2.5 B can damage the canal
min n	2.5	3.5	4.5	To keep on speed	3.5	5	7	To keep on speed
min h (over bottom width)	1.3 T		1.4 T	Considering squat & bow thruster efficiency	1.3 T		1.4 T	Considering squat & bow thruster efficiency
min R (ΔF needed for $R \neq \infty$)	4 L	7 L	10 L		4 L	7 L	10 L	
max v_{flow} (longitudinal)	0.5 m/s				0.5 m/s			

Table 2: Extract from the comprehensive table in the WG 141 report concerning the Concept Design Method for canals - "basic dimensions" for straight sections

This extension of the "basic dimensions" will be accounted for in the next subchapter (application of the Extended Concept Design). In our example of fairways in canals, extra widths will be specified for crosswind attack, cross currents from e.g. intake or outlet structures, higher vessel speed as assumed in Table 2 and a higher traffic density. Let's look here on the extra width in curves only:

The basic formula is $\Delta F_C = c_C \cdot L^2/R \leq L$, with ΔF_C = extra width of one vessel in a curve, c_C = a parameter, depending on the vessel type (the more "slender", the smaller c_C), the loading conditions (larger c_C for empty, smaller for loaded vessels) and the S&E category. L denotes the overall length of the vessel. In case of two-way traffic, the ΔF_C of both vessels must be added, considering e.g possibly different loading conditions or driving directions in case of longitudinal flow. Specific numbers of the parameter c_C will be provided in Appendix E, not only for canals, but also for rivers with its typical flow velocities and vessel types. For canals, concrete numbers will be recommended e.g. for Class Va vessels. For Class Va vessels and a S&E category better than C, the parameter c_C should be chosen to be 0.3/0.6 for loaded/empty vessels and 0.2/0.4 accordingly for Class Vb vessels, if significant longitudinal flow velocities of max. 0.5 m/s cannot be avoided. Analogous to the Dutch guidelines, the recommended standard manoeuvre in case of two-way traffic is an encounter of an empty vessel with a fully loaded vessel, if there are no other arguments speaking for a different traffic situation. This must be considered in choosing the c_C of each of both vessels.

The next subchapter of Chapter 5 resumes results from collected practice examples if applicable. This is important especially for fairways in rivers. In case of canals, practice examples are generally identical to the application of guidelines and will thus not be regarded.

Special considerations concerning the use of simulators follow in the next subchapter. In case of canals, several hints will be given to account for e.g. the strong increase in vessel resistance close to the critical ship speed (by assessing this speed limit with provided formulae and by recommending to stay significantly below during simulation runs) or the effect of strongly confined conditions on the decrease of bow thruster efficiency by the return current velocity, which must be added to the vessel speed. Because especially the strong and altered interaction of ship-induced wave fields in canals compared to shallow water seriously affects the driving dynamics of inland vessels in narrow canals, and because these effects are considered by semi-empirical formulae only in most simulators, basing in most cases even aggravating on shallow water data with some adaptions to sideways restricted waters, the application of the principle of comparative variant analyses is mandatory especially for canals. This is, to mentions this important point again, to reduce modelling inaccuracies as those from parameterized bank and ship-ship interaction forces as far as possible.

The last subchapter draws conclusions from all these considerations. In our example of canal design, the following statement may be cited: *"In most cases the Concept Design Method and the Extended Concept Design will be sufficient for the design of fairway in canals. Additionally a wide variety on guidelines covers these aspects of design. Only exceptional design situations demand a detailed study."* These are e.g. entrance and exit manoeuvres at lock approaches or turning manoeuvres.

5.6 Conclusions

This chapter outlines again inter alia the following points:

- The proposed design approach should be as objectively, rational and quantitatively as possible, leading to the Three Methods Approach with the core element S&E Approach.
- The design should be restricted to reasonable cases. This means e.g. that restrictions concerning the ease of navigation, e.g. in case of strong wind (the empty vessel may have to moor to let another less wind-sensitive vessel pass by) or seldom occurring traffic situations, if they can be avoided in practice, may be accepted.
- To restrict modelling expenses, the Detailed Design should focus on decisive design cases. The latter may be found by "scanning" all possible relevant variants with less simulation effort or by using the Concept Design Method where appropriate.
- Decisive design case should be run several times to account for e.g. human-related effects, using averages of relevant parameters and in comparison to reference cases. This comparison should be made as far as possible by using quantified criteria as those from the Detailed S&E Approach.
- Interpret results, especially from simulations, with care, because even the best nowadays available methods may be inaccurate in critical design cases. The inaccuracies may also come from flow models, e.g. because of unconsidered secondary currents using depth-averaged 2D models, which is often the case.

5.7 Appendix A: Summary of existing guidelines

The WG analysed guidelines from Belgium, China, The Netherlands (also called Dutch guidelines), France, Germany, Russia and the USA according to the a.m. waterway dimensions. As mentioned earlier, the WG members assigned adequate S&E categories to recommended figures and used averages of those with the same ease category as a basis for the recommended Concept Design Approach. Without going into details, comments to the chosen guidelines may be made as follows:

- Belgium guidelines consider basic dimensions of canals only (extension by Dutch guidelines).
- Chinese guidelines regard all waterway dimensions considered here, also rivers for different vessel categories and flow velocities. They offer numerous design formulae, e.g. for cross flow and lead, compared e.g. to the German or French guidelines, to a relative generous design.
- Dutch guidelines offer very comprehensive and detailed design rules. This concerns e.g. to traffic density or wind effects, which were adopted to a large extent in the WG 141 report.
- French guidelines lead, compared to the average of the other guidelines, to relatively "narrow" waterway dimensions, especially concerning fairways in rivers or bridge openings.
- German Guidelines consider waterway dimensions for canals only as in the Belgian and Dutch guidelines. Unique is how to compose fairway widths by different components as vessel breadths, extra widths due to human factor and the more precise consideration of curve increments.
- Russian guidelines were evaluated because numerous influencing parameters were considered as the influence of flow velocities at lock approaches.
- US guidelines lead to smallest fairway widths in large rivers compared to the other guidelines. Reasons are the generally very low operation speed of large US push tow units, together with highly powered and manoeuvrable pushers (2-wheeler).

As mentioned earlier, the national guidelines reflect the typical boundary conditions as fleet composition and waterway types with its unique water depths and flow velocities and the tradition of shipping in these countries. They lead to different S&E demands and thus, to different waterway dimensions. Note again that these differences were the starting point of the special S&E approach.

5.8 Appendix B: Dimensions of existing waterways - practice

The evaluation of practice data shows the partly huge differences in existing waterway dimensions. Looking e.g. on realized bridge opening widths (one bridge field): The averages of different waterways vary between 2·B (B = largest vessel breadth) and 6.6·B with averages over the different waterway considered of around 4·B for upstream and 2.9·B for a downstream drive. Averages of different waterways of lock approach widths show a similar variation, e.g. figures between 2.5·B and 4.9·B for double locks. This shows that, realized waterway dimensions are rather reflecting the "possibilities" of construction than the nautical "necessities". Hence, practice data should be interpreted with care, but they are – of course – very helpful for comparing and thus evaluating designed waterway dimensions.

5.9 Appendix C: Appropriate assessment of safety and ease quality and its usage for design

This appendix goes in more detail than Chapter 3 in the main report, especially concerning the way to quantify S&E using selected time series of relevant parameters from the results of simulation runs or the direct quantification of human influences and shall be outlined here in a bit more detail as follows:

As mentioned earlier, the report proposes the standardized NASA TLX-Test (Task Load Index) for this purpose. The index scales the "work load" by steering the vessel. The index can be used in the same way as the Detailed S&E Score to compare variants as the design case with the ease reference case. The test presupposes that the *same* pilot steers the vessel in the reference cases and the design cases as well. What he has to do, e.g. during the usual coffee breaks between two simulation runs, is, to answer six questions in a similar way as using the table of the Simplified S&E Approach, assessing a score between 0 and 1 for each question. If, for example, the question: "How insecure, discouraged, irritated, stressed and annoyed were you?" could be answered with "low", meaning, the pilot had everything under control even during a critical manoeuvre, the corresponding score is very low too and thus the work load. The different scores will be matched together using different weighting factors of each single score. The weighting factors result from comparison of six other criteria with each other by assessing, which criterion is more important in the direct comparison of 2 criteria, the one or the other. The answering of these questions may last about 5 minutes. This is less than the usual coffee break time between two drives in a ship handling simulator of about 15 minutes. So, the TLX test may not extend the usual time required to use SHSs and thus don't increase the expenses, but the results of the TLX test support, together with the other aforementioned criteria, the interpretation of simulated results.

5.10 Appendix D: Detailed or case by case design – using simulation techniques

In this appendix, the approach to perform an "ideal study" as outlined here in Chapter 5.4 and Figure 6 will be justified, explained and worked out in more detail. Note that these recommendations focus on waterway infrastructure design only. So, especially concerning the usage of SHSs, the report does not look on general demands to ensure its applicability as described in numerous existing and arising documents as the PLATINA II CESNI-report. It is rather presupposed that these demands are fulfilled and that the provider of SHS services checks the applicability of his simulator before preparing an offer for services and the client is responsible to ask for documents verifying the general applicability of the simulator. So, the report doesn't give answers on *what* a simulator is capable of – this is the objective of e.g. the CESNI-report –, only *how* its application can be checked (verification) and *how* it can be used optimally regarding the quantification of necessary waterway infrastructure dimensions.

5.11 Appendix E: Extended Concept Design Method – account for extra widths

As mentioned here in Chapter 5.5 by example of the fairway design in canals, the "basic dimensions" which are given in tables as those in Table 2, must be extended by so-called "increments" to account for special influences. Additionally to the recommendations given in the subchapters of Chapter 5 concerning the Extended Concept Design, Appendix E provides more information on that matter, e.g. formulae with wider application ranges, tables of calculated extra widths or corresponding parameters as the c_C in the formula for curve increments for practically relevant boundary conditions in canals and rivers. The approaches behind the formulae are comprehensively explained too.

As an example, the approach how to determine extra width to account for cross wind are explained and prepared for practical application. The corresponding subchapter is very comprehensive because the authors of the report recognized the lack of appropriate information on this subject. Therefore, information from existing guidelines, e.g. from Spain (inter alia concerning the wind profile and the gust factor), from The Netherlands (inter alia concerning the design wind speed), the USA (e.g. concerning crosswise wind forces and forces on the underwater body of the vessel while drifting), together with relevant standards to assess e.g. the influence of the roughness of the surrounding area on the vertical wind profile and gusts etc., were evaluated and matched together to end up with appropriate design formulae. The latter support the "Dutch Rule" for wind increments ΔF_W to be multiples (factor c_W) of the vessel length L ($\Delta F_W = c_W \cdot L$, $c_W = 0.05$ for inland waterway stretches for empty vessels).

The way to ascertain these numbers for various other conditions as different loaded container carrying vessels is illustrated in this paper in Figure 7. An extract of a large table in the report, containing calculated c_W, is presented here in Table 3 too. The calculations were made making (among others) the following assumptions:

- The reference wind speed for an inland stretch is 10.5 m/s according to the Dutch guidelines (5-6 Bft, measured in 10 m height and averaged over 10 minutes).
- This wind speed will be first reduced according to the wind-exposed height of the vessel, assuming a surface roughness of agricultural terrain (scrublands, heath or meadows, no windshield) and then increased again by 1 minutes lasting gusts (leading to 13.3 m/s for an empty vessels or vessels with 2 layers and 13.6 m/s for 3 layers) – valid for S&E quality A.
- The vessels sail in unidirectional traffic, compensating the wind forces by drifting.

The listed c_W in Table 3 show that, depending especially on the number of container layers and water depths, the extra widths may be larger than according to the Dutch 5% rule. Note additionally that the extra widths were calculated without bow thruster usage. The latter would reduce the c_W (safe side).

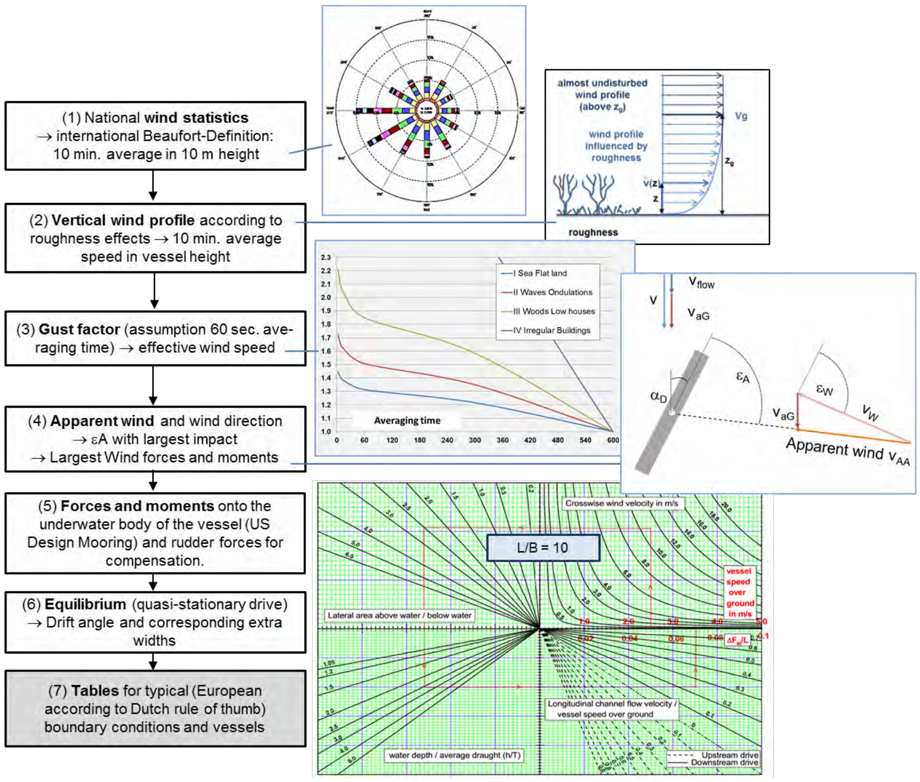

Figure 7: Visualization of the approach to determine extra width to drift against the wind

vessels and average draughts for empty vessels & 2 container layers (T2) respectively 3 layers of containers (T3)	flow velocities, waterway types, loading condition respectively number of container layers and corresponding wind-exposed height h_{SW}, driving direction							
	Canal, $v_{Flow} \leq 0.5$ m/s, acting always in driving direction: $v \approx 9$ km/h), $v_{aG} \approx 10.8$ km/h		$v_{Flow} \leq 1.0$ m/s $\approx$ **impounded river** $v_{Flow}/v = 0.4$, $v_{ag} \approx 5.4$ km/h upwards and 12.6 km/h downwards		$v_{Flow} \leq 1.5$ m/s $\approx$ **free flowing river**, $v_{Flow}/v = 0.4$ upstream and 0.5 downstream, $v_{aG} \approx 8.1$ km/h upwards and 16.2 km/h downwards			
	empty/ 2 layers h_{SW} = 4.5 m	3 layers h_{SW} = 7 m	empty/ 2 layers h_{SW} = 4.5 m	3 layers h_{SW} = 7 m	downstream drive		upstream drive	
					h_{SW} = 4.5 m	h_{SW} = 7 m	h_{SW} = 4.5 m	h_{SW} = 7 m
GMS (110x11.4, **Class Va**), T2 = 1.6m; T3 = 1.8 m	h = 4 m: **0.060**	h = 4 m: **0.08**	h=3 m: **0.05** h=6 m **0.08**	h=3 m: **0.06** h=6 m **0.11**	h=3 m: **0.04** h=6 m **0.06**	h=3 m: **0.05** h=6 m **0.08**	h=3 m: **0.02** h=6 m **0.04**	h=3 m: **0.03** h=6 m **0.05**

Table 3: Extra widths c_W due to drifting against the wind – Class Va vessel for different loading conditions (T2=empty vessel or 2 layers of partly loaded containers, T3 = 3 layers)

5.12 Appendix F: Application of the Detailed Design Approach to an example

In this appendix, the Detailed S&E Approach will be applied to nautical investigations on the River Danube by Real Time Simulations. The example follows strictly the recommended approach outlined here in Figures 5 and 6.

Referring e.g. to Figure 5, the check of a possible application of the Concept Design method was made in previous studies, showing that the existing fairway dimensions in the German free flowing Danube river stretch downstream Straubing are far away from recommendations in existing guidelines – and in narrow curves also from the figures given by evaluating practice data in the WG 141 report. So, the performance of a detailed study was inevitable. The "Modelling Capability Check" in Figure 6, to mention just another important step, was carried out by referring to previous nautical studies concerning the Danube River with the largest approved vessels, where data for comparing e.g. the swept area widths were available. The models used could thus be verified by observed field data. The reference cases refer simply to the existing situation, but for different water levels between average low water and highest navigable water. The investigation of different water levels and loading conditions as well were unavoidable because low up to mean water levels may be relevant especially in areas with small fairway widths as between spur dykes and buoys, and high levels may be relevant even if larger navigable widths are available, because the high flow velocities increase the navigation space needed, especially sailing downstream. Therefore, always three relevant water levels were considered, both for the reference and the design case. The latter refers to the Danube River after improvement by planned training measures. The investigations were carried out with a Full Bridge Simulator, because especially the human factor is very important in case of the stressful driving situations, especially while passing a very narrow bridge and a very narrow bend.

	Final assessment for curve km 2314			
	SV	RNW	MW	HNN
"erc"	Actual	60.7%	56.2%	53.9%
"dc"	Planned	61.7%	59.3%	53.9%

Figure 8: Selected results (numerical values = averaged navigational "reserves") of the application example in the WG 141 report, using the Detailed S&E approach

The simulation results were then quantified with the Detailed S&E Approach, whereby especially the way how to ascertain reasonable scores from relevant simulated parameters will be explained in detail. The study use so-called "reserves" for selected parameters as rudder angles, engine power, bow thruster use, distances to obstacles as bridge piers or buoys, path widths etc. The reserves are defined as follows: reserve = characteristic value (e.g. actual bank distance or actual rudder angle), minus some critical value (as the usual safety distance or the rudder angle, producing the maximum crosswise force), divided by a scaling parameter (as e.g. the available navigational besides a vessel at a centric ship path of a bridge opening or the maximum rudder angle by construction). The application example of the Danube River improvement shows impressively how the Detailed S&E Design Approach works and which explicit results can be achieved. It could be demonstrated that difficult driving situations, as the downstream drive in the narrow "Reibersdorf" curve (numbers in the table of Figure 8) can be made with about the same safety and ease quality after waterway improvement, leading to higher possible draughts (design case) compared to the existing situation (reference case).

6 Application example of the Concept Design

The recommended design procedure outlined especially in Chapter 5.4 of this paper will be applied in the following by an example. Without going into details concerning the formulae or the simulation technique used – this would go beyond the scope of this paper –, the main steps needed to obtain the results will be outlined. The example refers to very narrow fairway conditions in an impounded river. The approach recommended in the report will be used to check, whether larger, but modern vessels may be approved in future. This is a typical area of application of the WG 141 report. As shown in Figure 9, a narrow Spree-bend (curvature radius R ≈ 250 m) in the urban area of the government district of Germans Capital Berlin will be considered. The banks are almost vertical and the net width in draught depth is about 40 m in the narrowest reaches. The impounded Spree River has generally low flow velocities of about 0.3 m/s at mean water (canal situation), but around 0.7 m/s at mean high water (MHW), which will be considered here (≈ highest navigable water level HSW).

Nowadays, small push tow units, which are typical for the waterways around Berlin, with L = 91 m length and B = 9 m breadth are approved, but the most traffic results of course from passenger boats, but the adjoining locks would (geometrically) allow for navigation with Class Va vessels (L=110 m, B=11.4 m). So, the chosen design case refers to this vessel type, and, because of the larger swept area width in a narrow curve, to an empty ship sailing downstream in the direction shown in Figure 9.

Figure 9: Map and pilot's view on the German Spree River (government district, Berlin)

According to Figure 5, the design starts, after specification of the design case with its unique boundary conditions (driving situation: one-lane downstream, vessel type: Class Va, loading condition: empty, bathymetry: river stretch with vertical walls and sufficient depth at high water levels, flow field: small but significant flow velocity, wind exposure: not that much because of urban area), by looking on relevant national and international guidelines. In our example, the German Guidelines and several others are generally not applicable. They treat canals up to flow velocities of max. 0.5 m/s only and the curvature radius should be larger than 500 m. So, the application limits are exceeded both because of flow and curvature. But because the formulae given for extra widths in curves are also valid for smaller R and 0.7 m/s is not so far away from the "canal-limit" 0.5 m/s, the German Guidelines may be used at least for comparisons. They lead to a necessary width of approx. 49 m (almost half of it refer to the "basic with" including safety distances and the other half to the curve increment). This figure is without extra wind increments, which are not explicitly identified in the German guidelines. For comparison, the Dutch Guidelines lead to approx. 52 m (ca. 45 % refer to basic width and curve allowances and ca. 10 % to wind influence) if a "narrow profile" (low S&E quality) will be assumed. Other guidelines lead to about the same results. So, even if we assume canal conditions and follow corresponding guidelines, the existing navigable space would be too small for the new ship.

The next step is to use the WG 141 recommendations for Concept Design. For this purpose, the necessary S&E quality must be assessed first, using the approach outlined in Chapter 5.3 and illustrated for the application example in Figure 9. The corresponding table with criteria is scaled down in the middle of Figure 9. The main components of the table are two coloured columns with opposed arguments. The red coloured arguments or statements speak for a higher necessary ease quality and the other arguments in the 2nd column, which is marked in green colour, specify cases, where a lower ease quality may be acceptable. The applier of the Simplified S&E Approach has to check the truth content of both opposed arguments and assign a number (score) between +1, if the red argument is completely correct or -1, if the green statement is absolutely true, otherwise a score between them must be chosen, e.g. 0, if both arguments are true or neither the red or green argument, see step (1) on the left side in Figure 9. These single scores of in total 11 criteria have to be multiplied first with so-called "single factors", then with "group weighting factors" and finally added up, forming the comprehensive Simplified S&E Score. If there are doubts about this approach, e.g. about the group weights, the approach should be applied to a well-known driving situation, e.g. the present nautical conditions (called "Analysis Case" in the report with its own table) and compare the expected S&E category with the one from applying the approach. Maybe adapted weights where appropriate, but it must be assured that the sum of weights is always 1 – or the sum of scores has to be divided by the sum of weights, so that the entire score stays between +1 and -1. In our example, such an adaption was not necessary. To show how the scoring system works, let us look on the 7th criterion concerning the influence of the vessel equipment and instrumentation. It is displayed larger on top of Figure 9: Because the vessel is assumed to be empty and because the Waterway Authorities may demand for restrictive approval conditions for the vessel's equipment as strong bow thrusters and highly powered main thrusters as well as modern information systems, the single score is clearly -1, meaning, a lower ease quality may be acceptable because of this criterion.

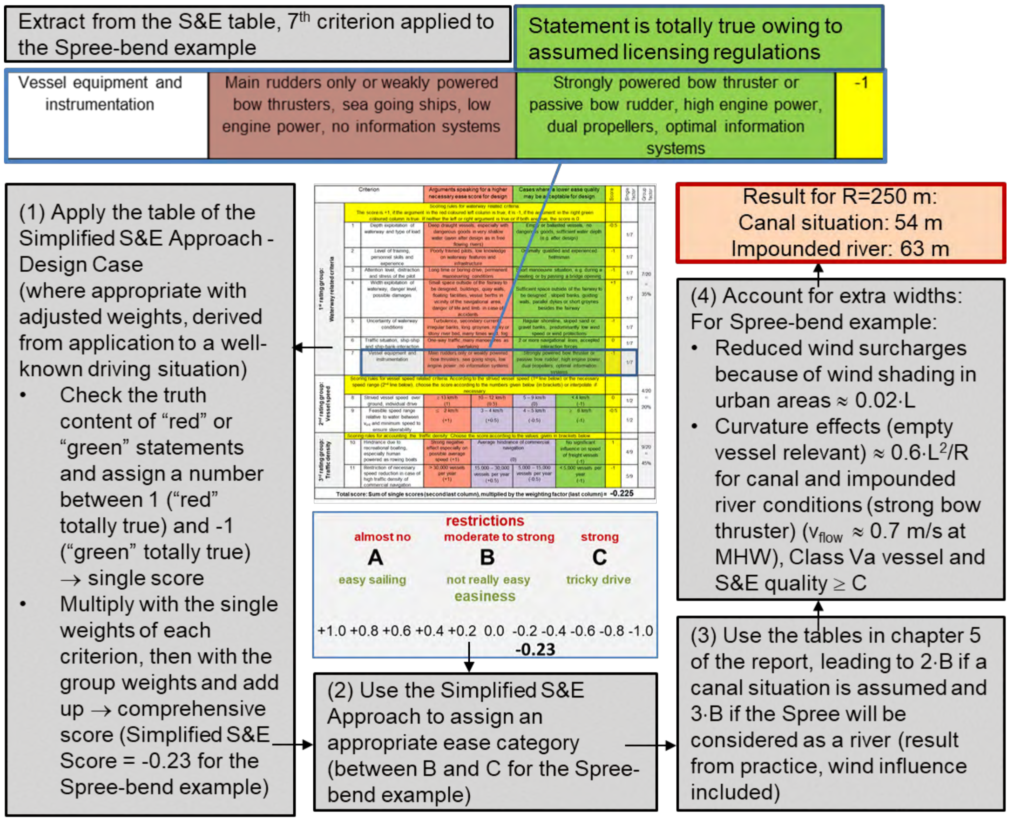

Figure 9: Flow chart concerning the Simplified S&E Approach to the Spree-bend example

The corresponding comprehensive Detailed S&E Score equals to -0.23, which can now assigned to an ease category, see step (2) in Figure 9. The report proposes for this purpose a graph, shown here below the table of criteria in Figure 9. It was "designed" by applying the approach to different examples and compare it to expected results. It turns out that the S&E is between B ("moderate to strong restriction", "not really easy sailing") to C ("strong restrictions", "tricky drive"). Now the Concept-Design of the report can be applied, e.g. using Table 2 of this paper, if canal conditions will be assumed, leading to a "basic width" of 2·B. This result is indicated in Figure 9 bottom right (step 3). If the Spree stretch will be will be considered to be an impounded river, the evaluation of practice data in the report leads to 3·B for a S&E quality between B and C. This figure includes extra widths due to wind attack. Now relevant increments have to be added to this basic widths, see step (4) in Figure 9. For a Class Va vessels without using bow thrusters, the figures in Appendix E of the report lead to c_C = 0.6 in a canal with significant longitudinal flow velocities up to 0.5 m/s. For an impounded river, c_C = 0.8 will be given. This figure may be reduced in case of strong bow thrusters as assumed here to again 0.6. So, the extra width in curves is for both cases – canal and river situation, equals to about 29 m. For the canal-case wind influences may be added too. Without proof here, the consideration of the larger roughness-effects in urban areas (lower average wind speed, but somewhat increased gust influence), reduce the factor c_W in of Table 3 of this paper of about 0.06 for agricultural terrain surrounding the waterway to only 0.02. The corresponding extra width is thus around 2 m only. In the end, the necessary fairway width totals up to 54 m in assuming canal conditions and 63 m for assuming an impounded river. Also this result shows that the 110 m long vessels may not be approved according to the results of Concept Design. It should be added that, the same approach, applied to the approved vessel types of the existing situation, fits with the existing width at the narrowest point of 40 m, if canal conditions will be assumed. So, the approval of larger vessels demand for a detailed study.

REFERENCES

The present paper refers mainly to the report of PIANC INCOM WG 141 on "Design Guidelines for Inland Waterway Dimensions", to appear probably summer 2018. Further references may be related papers submitted to the Smart Rivers Conferences of 2013 and 2015 and to the PIANC World Congress in 2014.

REMOTE CONTROLLED MARINE SECURITY OF LOCKS

By: Luc Boisclair, P. Eng

INLAND NAVIGATION CHANNELS: SAFETY AND RELIABILITY

This article provides insight into a working implementation of a remote controlled, state-of-the-art security access control and intrusion detection system at locks and bridges in an inland waterway.

The Story

The St. Lawrence Seaway Management Corporation *(SLSMC)* operate and maintains all Canadian Locks, Bridges and Channels that allow domestic and foreign vessels to navigate from the Atlantic into the Great Lakes. We manage 13 locks, 18 movable bridges, 13 km of approach walls, a power generation plant, fixed bridges and channels spread over 325 nautical miles (600 km). Our mission is to serve our customers by passing ships through a safe, secure and reliable waterway system in a cost effective, efficient, environmentally and socially responsible manner to deliver value to the North American economy.

In 2008, when the IMO (International Maritime Organization) ISPS (International Ship and Port Facility Security) code was implemented in Canada, the physical security of the Seaway's Canadian locks and bridges was improved mainly by installation of perimeter fencing, manual access control gates and basic PTZ (pan-tilt-zoom) surveillance cameras. Security responsibilities, including monitoring, were assigned to our locks personnel. Our Operations Control Center staff also had approximately 100 cameras available to validate security issues, although these were primarily used to support marine traffic operation.

In 2013, the Modernization Project was started, with the goal of increasing safety of operation, along with cost efficiency by 1) automating vessel mooring in order to remove the dangerous handling of steel mooring lines & 2) controlling our locks remotely from our Operations Control Centers. From 2013 to 2017, all locks were equipped with vacuum mooring technology along with controls and telecommunication for remote operation.

In 2015, improvements were required to our security posture during the re-assignment of our lock personnel to the Operations Control Center. Consequently, our Marine Security Project was created and integrated along with our modernization efforts. The objectives were to increase protection against unauthorized access, increase access control of our facilities and vessels, and improve the continuous monitoring of those facilities.

Within the next three years, the Marine Security Project would see to the implementation of the new security system at 13 locks, 7 free standing bridges and multiple other administrative, maintenance or support function buildings. Although we preferred at the time to use a design-build contractual approach, there was a need to quickly fix a few vulnerabilities. That civil work immediately created a fast track situation causing us to proceed with some in-house design followed by a main design contract and finally, a main supply and install contract with a national security company.

The Technology

Those requiring access include our own lock maintenance and operations personnel, pilots, mariners, vessel service providers, etc. Identification & access control smart cards are now issued by the Seaway for most regular users, while others can have their credentials validated remotely.

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Canada

For vehicle access control, dual motorized gates were installed and cameras were added in strategic locations to perform vehicle inspections. Drivers are now asked to step out of the vehicle and open doors to allow the remote operator to view contents of the vehicle.

For pedestrians, single person turnstiles with reporting stations have been installed. Pedestrian can now gain access by using their access card followed by a pin code entry or they have their identification validated by the remote operator who uses a camera mounted on the reporting station and an intercom to interact with the person requesting the access.

Additionally, non-automated access points are monitored through the use of an electronic key control system. Access to these manual gates, which are less frequently used, is done by first 'borrowing' the appropriate key from a key dispensing box. The key control system is integrated into our security software and allows the operator to track the older of the key.

Continuous monitoring is performed using thermal imagery cameras with intrusion detection video analytic software. This has been a challenging application to design and configure as the intrusion detection system has to cope with several moving targets such as movement of ships and of lock equipment, public vehicles, nearby pedestrians, wildlife, etc., in an outdoor environment. The thermal imagery intrusion detection is also complemented by break beams, motion detectors and conventional building door and window closure detection hardware.

Several PTZ (pan, tilt & zoom) IP (Internet Protocol) cameras were also installed at each of our locks to improve investigation capabilities and promptly respond to security incidents. PA (public address) systems were already present at some locations in a different capacity and were upgraded to have improved coverage at all locations.

In the Operations Control Center, the software for security and video systems form an integral part of the lock operators' workstations. The security system software's intrusion detection alarms automatically triggers camera placement and video playback loops within the video system software. Digital video recording is generated for all camera feeds, including thermal video.

This project also involved a substantial upgrade to our telecommunication and computing infrastructure. Local and inter-city network connections had to be improved for both bandwidth and latency to support all video, audio and control feeds. Additionally, we applied cyber security measures to our new security technologies. All central controls and networking cabinets are now physically protected and form part of the restricted area. Two factor authentications for operator stations have also been implemented. Both software for the security system and video control are now running from a fault-tolerant physically distributed virtual computing environment.

How it Works

Access control and security incident response activities are fully integrated into the lock operator's role. The security protocols are effectively applied without the need for a dedicated security department. For incidents that cannot be managed remotely, a roving team member is sent to investigate and/or if there is an imminent threat, local police is called. We also have maintenance personnel that can respond around the clock, interrupting their maintenance activities being conducted at a nearby structure or maintenance center.

The benefit is that the operator has constant situational awareness of anything which could impact the security of the transiting vessel. The operator knows who is at the lock and is in constant communication with the vessel master (pilot or captain) to ensure integrated security. Furthermore, this provides vessels with a more secure passage in our inland waterway system while also achieving our staffing optimization goals.

Since security tasks must be executed flawlessly, one main challenge is to ensure that we have sufficient staff to do both security and operations tasks in a variable staffing environment. When vessel traffic is light, the Operations Control Center is not fully staffed and when peak of work arise, staffing needs to be quickly ramped up. It is therefore critical to minimise false alarms and keep the security system operating at peak performance in order to avoid the generation of any additional and unnecessary burden to operators.

In the beginning, access control or intrusion detection response duty was considered a distraction to the operator from their core task of processing vessels. With change management, time and proper training, security tasks are now an integral part of normal operation, much like safety and transit efficiency.

This is not only a technology implementation project. The improved security posture is maintained through several layers of multifaceted security measures that includes improved physical security, social engineering, cyber security, all of which are interconnected. In order for the system to remain effective, the technology needs constant care and support.

The new security system was designed to cover the application of MARSEC Level 1 though it will also facilitate operation at MARSEC Level 2 without substantial additional staffing costs. At MARSEC Level 3, our operation would be temporarily shut down and as such there are not any specific provisions for MARSEC Level 3 in this project.

Lessons Learned

One success story was the test of the integration between the security system, the video analytic software and the video display software early in the design. Although all suppliers claimed interoperability, a certain amount of customization was required. Those requirements were defined ahead of time, which allowed us to shorten implementation time and costs.

On the constructive side, the following items were either unexpected or did not go as well as anticipated;

- Overall security system performance accountability: Holding the general contractor of the security project accountable for the overall performance of the system was not possible. Malfunctions and limitations could develop within any of the telecom, networking, computing or software layers, which we would internally be responsible to resolve. In addition, because of the environment, the security contractor did not want to commit to meeting a certain level of false alarm rate standards. As this issue extends into the future, it will be difficult to establish an in-service support and maintenance contract with a sole provider, along with a defined service level agreement.

- It was quickly realized that intrusion detection using video analytic processing of thermal imaging could not be designed and scoped with precision ‘on paper’. ‘Dead spots’ were identified only once the installation of the first set of cameras on a lock was completed, and thus, additional hardware was required to correct deficiencies through trial and error. Although equipment manufacturers and designers had software tools to evaluate the theoretical coverage, some modifications were in fact required. Several weeks of auto-learning mode and manual tweaking of the physical mounting, zone definition in the software, etc., was required to get the false alarm rate to a reasonable level. Some of these efforts are still required when seasons and/or weather conditions change. Every season brings various challenges, such as the angle of the sun, different types of precipitation and varying wildlife. Finally, as minor changes to technology occur, or changes to the environment are required, constant support from technicians is essential on a steady state mode, in order to ensure the false alarm rate stays within manageable levels.

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- This project involved several layers of complexity;
  - Two languages (French and English)
  - Two geographical regions with previous regional standards
  - Mixed technology starting points
  - Some local specific requirements
  - The need to keep existing systems in place while the new system was being rolled out through different go-live dates for each individual location.
  - With a fast paced technology development, some of the selected components were declared obsolete before being installed.
  - Evolving requirements: during the deployment of the new system, other ideas were being tested and designs were continuously being improved.

Conclusion

Although the project timeline had to be extended by one year, we were able to provide the basic functionalities on schedule to support the remote control of the locks. Our Marine Facility Certificate of Operation has recently been renewed for 5 years; a testimony of our capacity to meet regulation requirements in the new operating model.

Furthermore, this project provides vessels with a more secure passage into our inland waterway system, while also achieving our staffing optimization goals.

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Physical model research on breaking logs for through the gate filling of new Sint-Baafs-Vijve lock

by

Jeroen Vercruysse[1], K. Verelst[1], T. De Mulder[2] and R. Timmermans[3]

ABSTRACT

The lift height of most inland navigation locks in the Flemish region of Belgium is limited to 2-3 m. For these locks openings integrated in the lock gate sealed by vertical lift valves or butterfly valves are commonly used as lock levelling system. To improve the spreading and energy dissipation of the filling jets and hence reduce the hydrodynamic forces on the moored ships, breaking logs (also referred to as energy dissipation bars) might be mounted at the downstream side of the gate openings. Beem et al. (2000) provide some Dutch design guidelines. Since the shaping of a gate opening across the thickness of a steel gate and the integration of the valves are somewhat country-specific, it was decided to set up a generic physical model at Flanders Hydraulics Research (Antwerp, Belgium) aiming at determining the effect of breaking logs on the flow inside the lock chamber and optimization of the breaking log configurations adopted in Flanders (Verelst et al., 2016). In this contribution, an account will be given of the specific research carried out in this model during the design of the levelling system of the new lock of Sint-Baafs-Vijve (river Lys, Belgium). During the physical model research, 4 different configurations were tested. For reference purposes, the first configuration did not have any breaking logs. Next, three configurations with respectively 7, 5 and 3 breaking logs were tested. At first the discharge coefficients of the configurations were determined, for valve openings ranging between 20 % and 100 % of the total valve lift height. It turned out that the influence of the breaking logs on the discharge coefficient was negligible for valve openings below 50 % and limited for higher valve openings. Secondly, the effect of breaking logs on the energy dissipation was studied. When adding breaking logs, the spreading of the filling jet increased and the maximum velocity reduced to approximately 60 % of the velocity measured in the core of the jet compared to the configuration without breaking logs. The research revealed that the exact positioning of the breaking logs with respect to the gate opening at the upstream skin plate is more important for the spreading of the filling jets than the amount of blockage of the gate opening at the downstream skin plate. The lowest velocities were achieved with the configuration with 3 breaking logs, which is the configuration with the least blockage of the gate opening at the downstream skin plate.

1. INTRODUCTION

Navigation locks are key structures for navigation in canals and canalized rivers, as well as for the accessibility of ports. In Flanders, the northern part of Belgium, new CEMT class Vb locks are being built in the canalized river Lys, e.g. at Sint-Baafs-Vijve, in the framework of the Seine-Scheldt project. Also in other canalized rivers, like e.g. the Upper Scheldt river and the river Dendre, lock upgrading is on-going. The lift height of all these locks is limited to 2-3 m. For these inland navigation locks with a limited lift height, a leveling system with openings integrated in the lock gate, sealed by vertical lift valves or butterfly

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valves, is commonly used in Flanders. These leveling systems are characterized by concentrated filling jets entering the lock, generating relatively high forces on the ship in the lock chamber. Breaking logs (also referred to as energy dissipation bars) might be mounted at the downstream side of the gate openings, to enhance the spreading and energy dissipation of the filling jets, hence reducing the forces on the ship moored in the lock chamber. Beem *et al.* (2000) provide some Dutch design guidelines with respect to breaking logs. Since the shaping of a gate opening across the thickness of a steel gate and the integration of the valves are somewhat country-specific, it was decided to set up a generic physical model at Flanders Hydraulics Research (Antwerp, Belgium) aiming at determining the influence of breaking logs on the flow inside the lock chamber and optimization of the breaking log configurations adopted in Flanders (Verelst *et al.*, 2016). This paper will give an account of the specific research carried out in this physical model for the design of the new lock located at Sint-Baafs-Vijve on the river Lys. The major aim of the scale model research is to evaluate the influence of breaking logs on the flow pattern inside the lock chamber and to optimize the geometry of the breaking logs for this particular geometry of the gate opening. The secondary aim of the scale model research is to determine the discharge coefficient of the gate openings and the spreading rate of the jet immediately downstream of the opening, to use this information as input parameters of the software tool applied during the hydraulic design of the filling and emptying system.

The outline of this paper is as follows. Section 2 describes the new lock of Sint-Baafs-Vijve and the filing and emptying system of this lock. An overview of the physical model, the measurements techniques used and the tested configurations is presented in section 3. The influence of the breaking logs on the discharge through the gate opening is discussed in section 4. For analysing the effect of breaking logs on the flow pattern in the lock chamber both visualizations with dye and velocity measurements are performed. Section 5 describes the results of the visualization of the flow pattern with dye while section 6 presents the measured flow pattern downstream of the lock gate. The conclusions of this paper are provided in section 7.

2. NEW LOCK OF SINT-BAAFS-VIJVE

Within the frame work of the Seine-Scheldt project the river Lys is upgraded to CEMT class Vb. Therefore, the existing lock of Sint-Baafs-Vijve with a length of 152.5 m and a width of 16.0 m has to be replaced by a new lock with a length of 260.6 m and a width of 16.0 m. An planview of the new lock chamber is presented in Figure 1. The design lift height is 3.22 m, computed between an exceptional high level in the upstream reach and the reference level in the downstream reach of the river. The water depth in the lock chamber, with respect to the floor, ranges between 4.70 m (when leveled with the downstream reach) and 7.92 m (when leveled with the upstream reach). The lock gates are of the mitre gate type. Intermediate lock gates are present to enable levelling of smaller ships with a part of the lock chamber, in order to reduce water consumption. The upper, intermediate and downstream lock gates are identical, consequently only one spare set of mitre gates is needed.

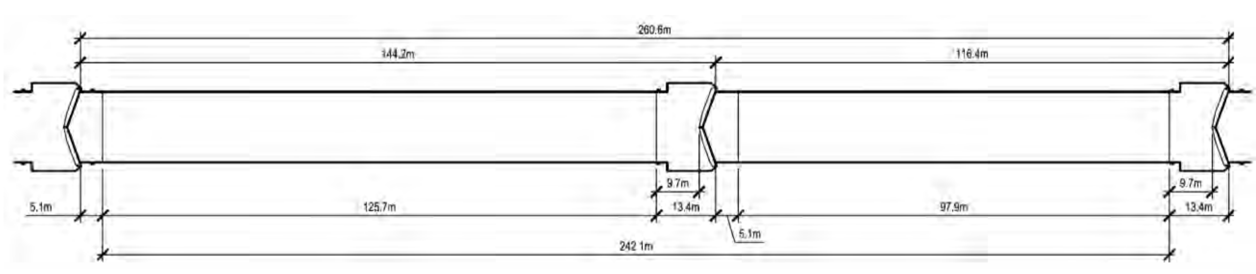


Figure 1: Planview lock chamber new lock Sint Baafs-Vijve

A frontal view from downstream, as well as a horizontal and vertical cross-section of the gate, are presented in Figure 2. The horizontal and vertical cross-section are situated at the center of the middle opening in the gate.

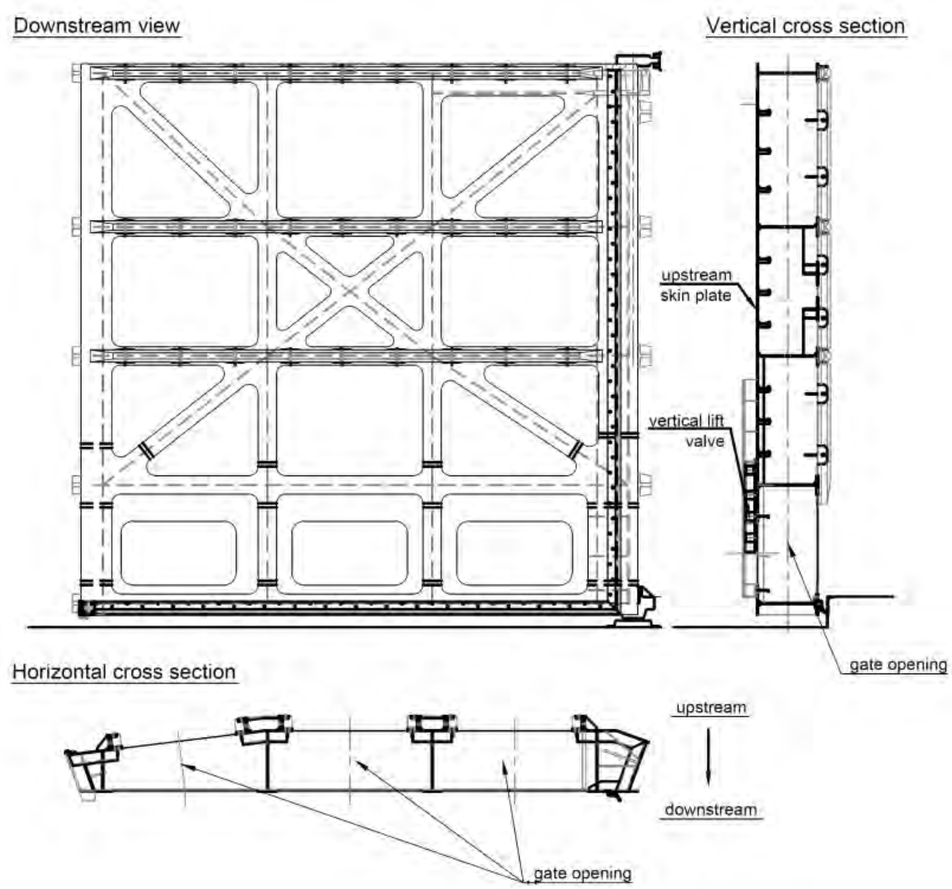


Figure 2: Mitre gate leaf with three leveling openings

The gate is constructed with both a vertical and a horizontal framing, as well as two diagonal struts. The thickness of the gate is 1.00 m. The gate is sealed at the upstream side with a closed skin plate. At the downstream side, a top plate is welded against the vertical, horizontal and diagonal girders. The leveling system is integrated in the mitre gates and consists of 3 rectangular openings in each mitre gate leaf placed at the bottom of the gate, hence 6 levelling openings per lock head are present. Each leveling opening is sealed with a vertical lift valve, mounted on top of the upstream skin plate. Each gate opening is delineated by the horizontal and vertical beams of the gate. These beams create a box with internal dimensions 2.69 m x 1.92 m (width x height). At the upstream side of the box, the gate opening has cross-sectional dimensions 1.89 m x 1.05 m (width x height), while the cross-sectional dimensions of the opening at the downstream side of the box are 2.40 m x 1.64 m (width x height).

During the physical model research, 4 different configurations of breaking logs are tested. The definition of the tested configurations was supervised by both hydraulic engineers and structural engineers involved in steel gate design, in order to be able to assess the hydraulic performance of a given configuration, as well as its (dis)advantages in terms of structural realization and maintenance of the gate.

For reference purposes, the first configuration (denoted C1) did not have any breaking logs. Next three configurations with breaking logs were tested: a configuration (denoted C2) with 7 breaking logs with section 0.15 m x 0.15 m and a distance between the breakings logs of 0.18 m, a configuration (denoted C3) with 5 breakings logs with section 0.15 m x 0.15 m and a distance between the breaking logs of 0.30 m and a configuration (denoted C4) of 3 somewhat larger breaking logs, section 0.20 m x 0.20 m, and a distance between the breaking logs of 0.30 m. For configuration C2 and C3 the upstream face of the breaking logs is positioned at 0.84 m with respect to the upstream face of the inlet opening. Due to structural reasons the breaking logs of configuration C4 are positioned 0.10 m more upstream then for the other configurations, i.e. the upstream face of the breaking logs is positioned at 0.74 m with respect to the upstream face of the inlet opening. The geometry of the four tested configurations is presented in Figure 3.

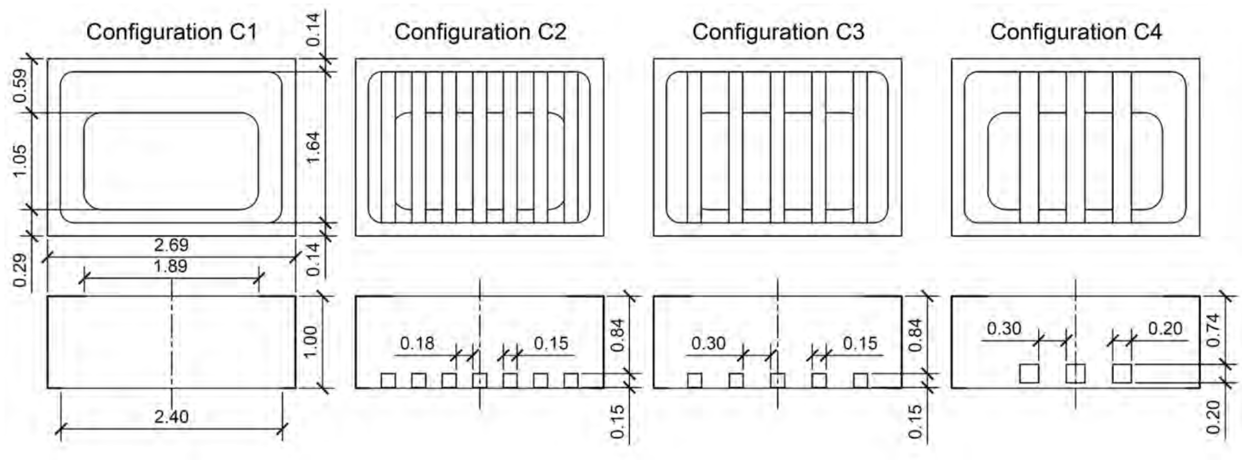


Figure 3: Tested configurations

With respect to energy dissipation bars (also referred to as breaking logs) the Dutch design manual (Beem *et al.*, 2000) presents the following three guidelines:

- Guideline 1: The blocking by the energy dissipating bars should be at least 50% of the downstream area of the opening without energy dissipating bars.
- Guideline 2: The total downstream opening, i.e. between the energy dissipating bars, should be larger than the total upstream opening in the lock gate. The largest flow velocity should occur in the gate and not outside the gate.
- Guideline 3: The smallest opening between the energy dissipating bars should be at least 0.30 m. Floating waste gets trapped in too narrow gaps.

Table 1 analyses how the different tested configurations of breaking logs meet the guidelines presented in the Dutch design manual (Beem *et al.*, 2000).

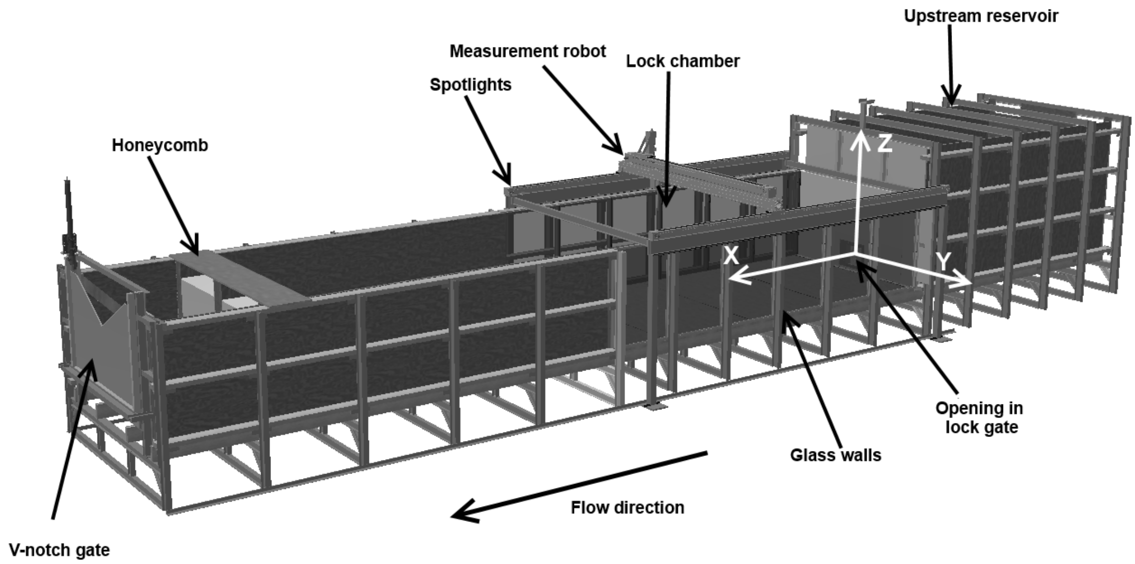
		Configuration			
		C1	C2	C3	C4
Cross-section upstream side of gate opening	[m^2]	1.98	1.98	1.98	1.98
Width of breaking logs	[m]	/	0.15	0.15	0.20
Number of breaking logs	[-]	0	7	5	3
Cross-section breaking logs	[m^2]	0.00	1.72	1.23	0.98
Cross-section between the breaking logs at downstream side of gate opening	[m^2]	3.94	2.21	2.71	2.95
$\frac{\text{Cross-section breaking logs}}{\text{Cross-section downstream side of gate opening}}$	[-]	1.00	0.44	0.31	0.25
$\frac{\text{Cross-section between the breaking logs at downstream side of gate opening}}{\text{Cross-section upstream side of gate opening}}$	[-]	1.98	1.12	1.36	1.49
Distance between breaking logs	[m]	/	0.18	0.30	0.30
Guideline met? (Beem *et al.*, 2000)	Guideline 1	/	No	No	No
	Guideline 2	/	Yes	Yes	Yes
	Guideline 3	/	No	Yes	Yes

Table 1: Characteristics tested breaking logs configurations

The blockage of the breaking logs ranges from 25 % to 44 %. Note that neither of the three tested configurations meets the first guideline, e.g. the blockage of the downstream side of the gate opening by the energy dissipation bars has to be at least 50 %. The three tested configurations with breaking logs meet the second guideline, i.e. the cross-section of opening at the downstream side of the gate between the breaking logs has to exceed the cross-section of the upstream side of the gate opening. Configuration C3 (5 breaking logs) and configuration C4 (3 breaking logs) meet guideline 3, i.e. the distance between the breaking logs is 0.30 m or larger, while configuration C2 does not.

3. **PHYSICAL MODEL**

The research was performed in a dedicated physical model for studying the influence of breaking logs on a lock levelling system with openings in the lock gate (Verelst et al., 2016, Ramos et al., 2016). A 3D sketch of this scale model is presented in Figure 4. The local reference system used in this paper is also indicated in white in this figure. The origin of this local reference system is situated at the center of the opening at the downstream side of the gate.



The local reference system used in this paper is indicated in white

Figure 4: Physical model

The physical model represents a part of the upstream reach and a part of the lock chamber with in between the lock gate. To reduce the complexity of the model, only a gate perpendicular to the lock chamber and only one gate opening in the lock gate is considered. Consequently only one filling jet is entering the lock chamber. Also no vessels are present in the lock chamber. The width of the scale model is 2.0 m, the length of the measurement section in the lock chamber is 3.75 m and the gate has a thickness of 0.12 m. At 7.9 m downstream of the gate a honeycomb structure is present, to smoothen the water surface in the downstream reach. The water level of the upstream reach is regulated by adjusting the supply discharge, while the water level in the downstream reach is regulated by an adjustable V-notch gate. To enhance flexibility of the set-up, the leveling opening in the model can be readily exchanged. For a certain opening in the lock gate a variety of breaking logs configurations can easily be installed.

During the physical model tests, the water level in the upstream reach and the water level in the lock chamber is measured using micropulse tranducers (from Balluff). The discharge feeding the upstream reach is measured by an electromagnetic flow meter (from Krohne), mounted on the supply pipe. The V-notch gate at the downstream end of the model is used as a complementary discharge measurement. Flow velocity measurements (see section 6) were carried out by means of an ADV velocity meter (Vectrino Profiler, from Nortek), making only use of the measurements in the “sweet spots” (in order not to suffer from the bias problems reported and explained in Thomas *et al.*, 2017). The specifications of the Vectrino Profiler give a maximum range of +/- 3.0 m/s. This range is not exceeded by the theoretical maximum velocity of 2.7 m/s. During tests, however, it was noticed that (due to fluctuations in the measured signal) higher velocities occur, but these are cut of at 2.8 m/s. For measured velocities above 2.8 m/s aliasing occurs, resulting in negative velocities. To correct this, a de-aliasing script is applied (Thomas et al., 2017). By attaching the Vectrino Profiler to an automated traverse system a dense grid of point velocities was measured. For visualizing the flow pattern, dye (Potassium Permanganate) is injected upstream of the inlet opening using simultaneously a horizontal and vertical half circular dye injector. With an underwater camera it was controlled that also the outer streamlines of the flow through the gate opening contain dye. A system equipped with 4 cameras (IDS UI-5420 RE), a power supply, data storage and 2 computers for data acquisition, is used to obtain sequential photos of the dye spreading. To provide a proper illumination of the measurement area 14 halogen spotlights with 90 W each are used, from which 12 spotligths are placed behind the left glass wall and 2 are used above the water surface.

Taking into account the available space in the physical model, a geometrical scale factor of 1:8.5 was used to scale the four tested configurations presented in Figure 3. Compared to the gate in prototype, the opening in the gate in the physical model is situated 0.049 m (in scale model dimensions or 0.417 m in prototype dimensions) too high above the floor of the physical model.

It should be noted that in the remainder of this paper all results will be presented in a dimensionless format.

4. DISCHARGE COEFFICIENTS

For the hydraulic design of a navigation lock the discharge coefficient is used to calculate the discharge through the opening in the lock gate in function of the water levels and the relative valve opening. The discharge through the gate openings is calculated using following formula:

$$Q = \mu A \sqrt{2g\Delta h} \tag{1}$$

where Q = discharge [m³/s]; μ = discharge coefficient [-]; A = is the cross-sectional area of the opening under the valve [m^2], g = the gravitational acceleration [m/s²] and Δh = the head [m].

The discharge coefficient expresses the efficiency of the levelling opening. The theoretical maximum value of the discharge coefficient is equal to one, indicating that no contraction and energy losses are present. Due to contraction and energy losses the actual discharge coefficient will be less.

During the physical model research, the discharge coefficient is (steady state) measured for different positions of the vertical lift valve, yielding a relative opening ranging between 20 % till 100 % (with a step size of 10 %). After the flow reached perfectly steady state conditions, the discharge, upstream water level, and downstream water level were recorded during a period of 360 s. For each test, the value of the discharge coefficient is computed as a time-average of the instantaneous values during this recording period. Figure 5 and Table 2 present the variation of the computed discharge coefficient for the four tested configurations in function of the relative valve opening. Note that the discharge coefficient is determined by using the discharge measured with the electromagnetic flow meter mounted on the supply tube, as well as by using the discharge determined from the overflow height over the downstream V-notch gate. The difference between the discharge coefficient calculated with the electromagnetic flow meter and the V-notch gate is for most of the measurements limited to +/- 1 %. For 3 measurements, however, the difference is somewhat higher and amounts up to -4 %.

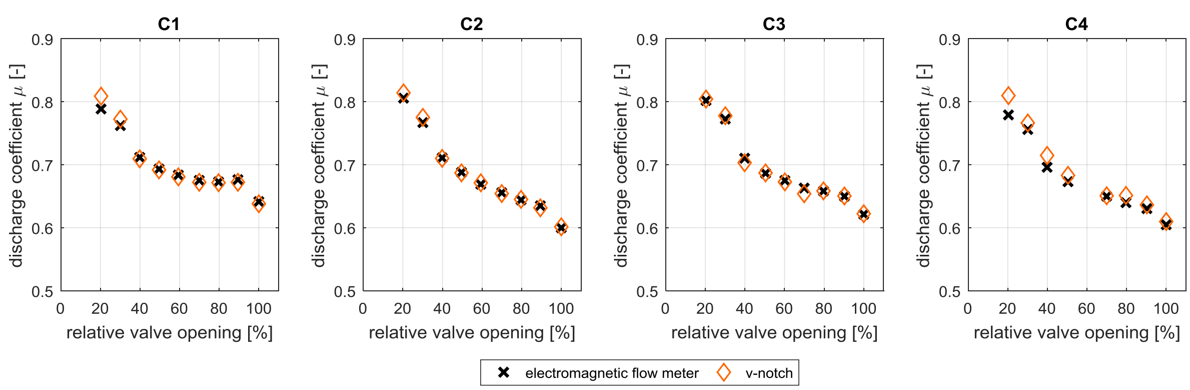


Figure 5: Discharge coefficient in function of relative valve opening

Relative valve opening [%]	Discharge coefficients											
	C1			C2			C3			C4		
	flow meter	V-notch gate	Δ [%]	flow meter	V-notch gate	Δ] [%]	flow meter	V-notch gate	Δ [%]	flow meter	V-notch gate	Δ [%]
20	0.79	0.81	-3	0.81	0.81	-1	0.80	0.80	0	0.78	0.81	-4
30	0.76	0.77	-1	0.77	0.78	-1	0.77	0.78	-1	0.76	0.77	-1
40	0.71	0.71	0	0.71	0.71	0	0.71	0.70	1	0.70	0.71	-3
50	0.69	0.69	0	0.69	0.69	0	0.69	0.69	0	0.67	0.68	-1
60	0.68	0.68	0	0.67	0.67	0	0.67	0.67	0	0.66[1]	0.67[1]	-1
70	0.68	0.67	0	0.66	0.65	0	0.66	0.65	1	0.65	0.65	0
80	0.67	0.67	0	0.64	0.65	0	0.66	0.66	0	0.64	0.65	-2
90	0.68	0.67	1	0.63	0.63	1	0.65	0.65	0	0.63	0.64	-1
100	0.64	0.64	1	0.60	0.60	0	0.62	0.62	0	0.60	0.61	-1

[1] Interpolated value.

Table 2: Discharge coefficient in function of relative valve opening

The discharge coefficient decreases with an increasing valve opening. Table 3 presents the influence of breaking logs on the discharge coefficient. The influence of the breaking logs on the discharge coefficient is negligible for relative valve openings below 50 %, i.e. the maximum difference is 2 %. For the lowest measured relative valve opening (20%) the discharge coefficient equals 0.80 (+/-0.02) for the four tested configurations. For relative valve openings above 50 % breaking logs result in a reduction of the discharge coefficient between 1 % and 7 %. For the vertical lift valve fully opened the discharge coefficient for the configuration without breaking logs amounts 0.64, while for the configurations with breakings logs a discharge coefficient of 0.60 is obtained for the configuration with 7 breaking logs (C2) and the configuration with 3 breaking logs (C4) and a discharge coefficient of 0.62 is measured for the configuration with 5 breaking logs (C3).

Relative valve opening [%]	Relative difference Δ compared to C1		
	C2 [%]	C3 [%]	C4 [%]
20	2	2	-1
30	1	1	-1
40	0	0	-2
50	-1	-1	-3
60	-2	-1	-3
70	-3	-2	-4
80	-4	-2	-5
90	-6	-4	-7
100	-6	-3	-6

Table 3: Influence of breaking logs on the discharge coefficiënt (based on the electromagnetic flow meter)

Prior to the tests for a rectangular opening a circular opening based on a previous design for the new lock at Sint-Baafs-Vijve was tested in the FHR physical model (Ramos *et al.*, 2016). During these tests the discharge coefficient was measured for relative valve openings from 10 % till 100 % for a configuration without breaking logs and for a configuration with three breaking logs placed inside or outside the gate opening. Van der Ven *et al.* (2015) compared CFD simulations and physical model tests for a geometry with two rectangular openings placed side by side in a flume with and without breaking logs. In the physical model tests the discharge coefficient is determined for relative valve openings 10 % till 100 %. Figure 6 compares the discharge coefficient presented in Figure 5 and Table 2 (based on the electromagnetic flow meter) with the results for the circular opening of Sint-Baafs-Vijve and the physical model tests described in Van der Ven *et al.* (2015).

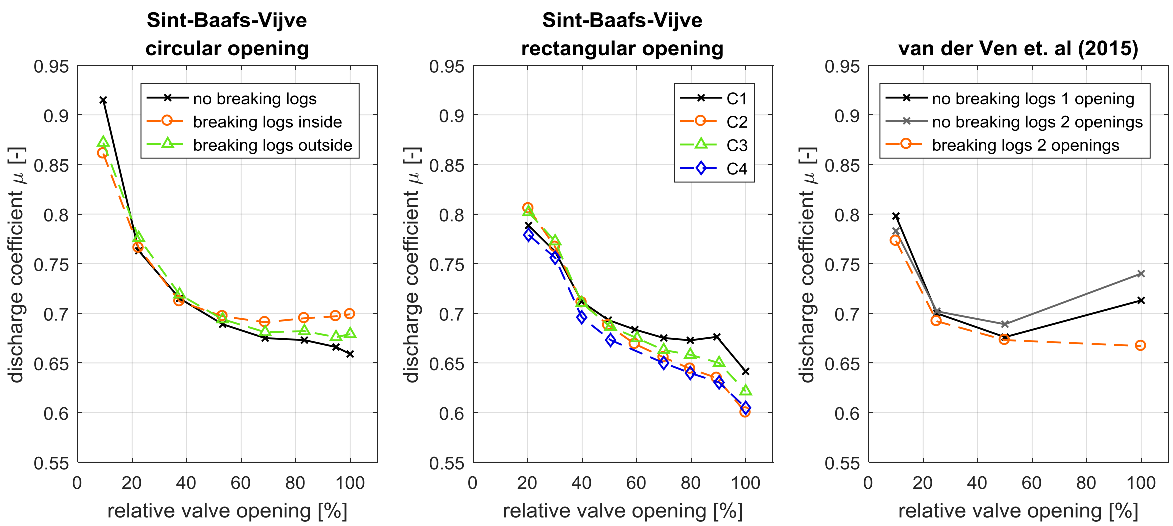


Figure 6: Comparison discharge coefficient for different physical model tests

Based on the comparison of the different physical model tests its concluded that for relative valve openings below 50 % the discharge coefficient increases. The effect of breaking logs on the discharge coefficient is negligible for relative valve openings below 50 %. At relative valve openings above 50 % the trends between the different physical model tests differ somewhat. For the circular opening with breaking logs the discharge coefficient is more or less constant for relative valve openings superior to 50 %. Without breaking logs the discharge coefficient decreases with an increase of the relative valve opening. When the valve is fully opened the discharge coefficient increases with 3 % when breaking logs are placed inside the gate opening and with 6 % when breaking logs are placed outside the gate opening. The tests described in van der Ven et al. (2015) show a contrary behavior for valve openings superior to 50 %, i.e. the discharge coefficient for tests without breaking logs increases with increasing valve opening. Note that when using 1 opening instead of 2 openings the discharge coefficient decreases with 4 % for the situation with fully opened valves. Adding breaking logs reduces the discharge coefficient with 10 % (for a fully opened valve and 2 openings present).

The manual of the software Lockfill (Deltares, 2015) recommends the use of values between 0.60 and 0.90 for the discharge coefficient. The discharge coefficients obtained from the present physical model tests are well within this range. The value 0.90 corresponds to the lower relative valve openings and the value 0.60 to the higher ones.

5. VISUALIZATION OF FLOW PATTERN IN LOCK CHAMBER

The jet in the lock chamber is visualized by injecting dye upstream of the gate opening. A top view and a side view of the flow pattern in the lock chamber is presented in Figure 7 for the experiments with the vertical lift valve fully opened and in Figure 8 for the experiments with the vertical lift valve half opened. During the different experiments with dye, both the color and the transparency of the water in the physical model varied. The contrast and color of each individual picture was adapted by a Matlab post processing script.

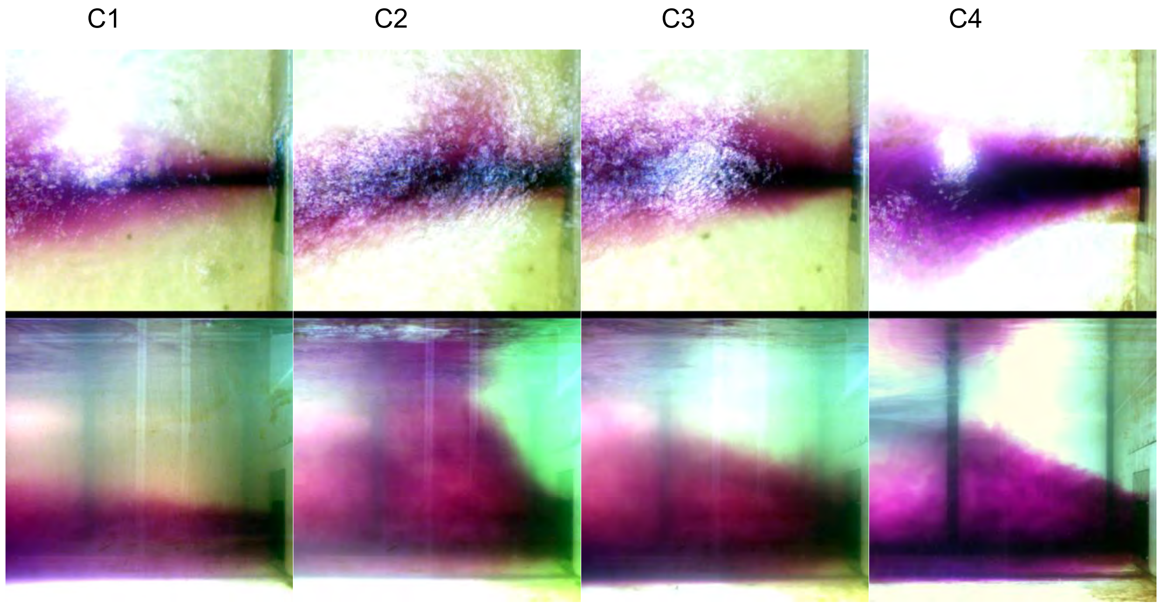


Figure 7: Visualized flow pattern vertical lift valve fully opened; above: top view; below: side view

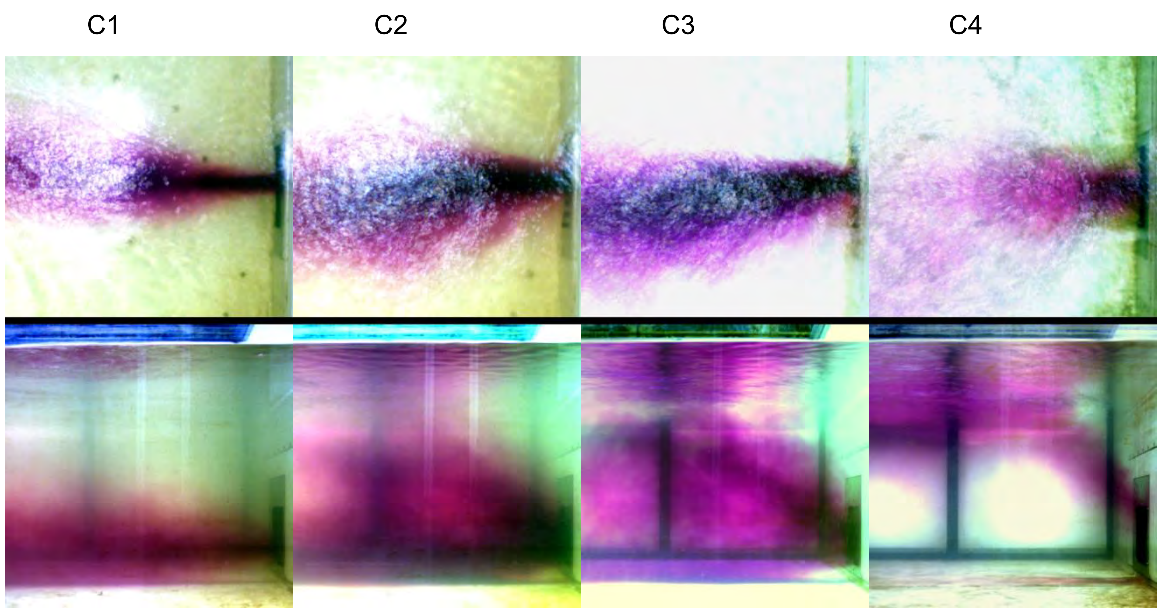


Figure 8: Visualized flow pattern vertical lift valve half opened; above: top view; below: side view

The figures illustrate that the (horizontal) breaking logs enhance the spreading of the jet in vertical direction. Although less apparent, also an increase of spreading in the width of the jet can be noticed when adding breaking logs. In the top view visualizations the color in the central part of the outflow is fully saturated. This central part is surrounded with a less saturated zone that makes the transition with the surrounding non colored water. The less saturated zone indicates the entrainment of ambient fluid with the fluid in the jet. Although both the top view picture and the side view picture are taken at the same time, a clear transition between a fully saturated and a less saturated zone is not detectable in the side view. The top view of the configuration with 7 breaking logs (C2) and the configuration with 5 breaking logs (C3) show that, both for the fully opened and the half opened valve, the filling jet is not passing the outer breaking logs. Therefore, a new configuration with 3 somewhat wider breaking logs was defined (C4). For the fully opened valve, the configuration with 3 breaking logs (C4) is comparable to the configuration with 5 breaking logs (C3), both in terms of spreading in the horizontal direction and in the vertical direction. The configuration with 7 breaking logs (C2) results for the fully opened valve in an increased spreading in vertical direction, i.e. towards the water surface, as compared to the other configurations with breaking logs. For the configuration with 3 breaking logs (C4) and the half opened valve, the lack of dye in the lower part of the water column is probably caused by a malfunctioning of the dye injection system. For the configuration with 3 breaking logs (C4) the spreading of the jet in the vertical direction to the water surface with the vertical lift valve half opened is comparable as for the configuration with 5 breaking logs (C3). Compared to the fully opened valve, the configuration with 5 breaking logs (C3) and the configuration with 3 breaking logs (C4) show a higher spreading in vertical direction, i.e. towards the water surface.

6. **VELOCITY FIELDS**

One of the main goals of the physical model research is to determine the effect of breaking logs on the velocity field downstream of the gate opening. Therefore, velocity measurements were performed in a dense grid of points in the lock chamber downstream of the gate opening. A first series of grid points is situated in a vertical plane at x = 1.7 m downstream of the gate opening (for reasons of comparison with previous research for a different gate opening geometry). A second series of grid points is situated in the vertical symmetry plane of the physical model.

During leveling of a lock the vertical lift valves are gradually opened. From the hydraulic design of the levelling system follows that the vertical lift valve is not fully opened at the moment of the maximum force on the ships in the lock chamber. Therefore, the velocity measurements are both carried out for the fully opened valve and for the half opened valve. Earlier velocity measurements in the physical model indicated a meandering behavior of the filling jet with a period of approximately 60 s (Verelst *et al.*, 2016) and an off-centerline excursion increasing with the distance from the gate opening. As a compromise between accuracy and available time, a measurement duration of 180 s (equal to approximately 3 periods of the low-frequency oscillations of this cyclic behavior of the jet) is considered for the velocity measurements.

For the visualization of the measured velocities in Figure 9 to Figure 11, the x-, y- and z-coordinates are presented in dimensionless form. As a length scale for the x- and z-coordinates, the height H (=1.05 m) of the (fully opened) opening at the upstream side of the gate is used, whereas the width W (=1.89 m) of the opening at the upstream side of the gate is chosen as the length scale for the y-coordinates.

Also the velocities are presented in dimensionless form, using the cross-sectionally averaged velocity U_0 in the opening (below the lower edge of the vertical lift valve) at the upstream side of the gate as the velocity scale.

Since $U_0 = Q/A$, from equation (1) follows:

$$U_0 = \mu \sqrt{2g\Delta h} = \mu \, V_{max} \tag{2}$$

where V_{max} denotes the theoretical maximum (cross-sectionally averaged) velocity [m/s] in the opening, if contraction and energy losses were absent. Note that the dimensionless value of V_{max} is given by:

$$\frac{V_{max}}{U_0} = \frac{1}{\mu} \tag{3}$$

which yields a value of about 1.56 for a discharge coefficient of 0.64 (section 3).

The measured velocities in the vertical plane downstream of the opening are presented in Figure 9, whereas the measured velocities along the vertical symmetry plane of the model are presented in Figure 10. Both figures present the velocities for the fully opened valve, as well as for the half opened valve. To reduce the number of measurements, at the elevations z/H = -0.65, z/H = -0.41 and z/H = +0.41 velocity measurements are only carried out for half of the model domain.

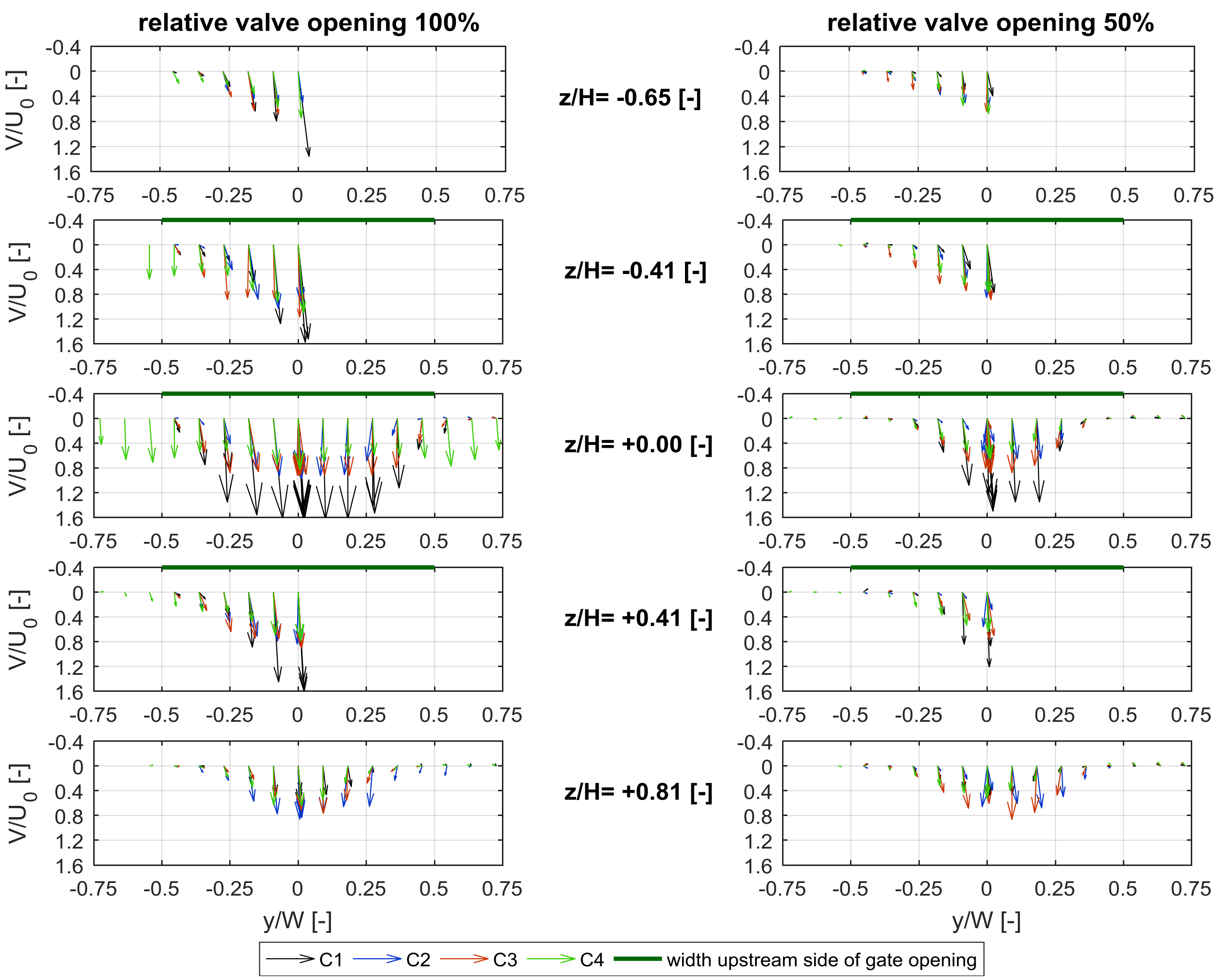


Figure 9: Horizontal velocity patterns at a distance x/H = 1.6 downstream of the opening and at different vertical elevations

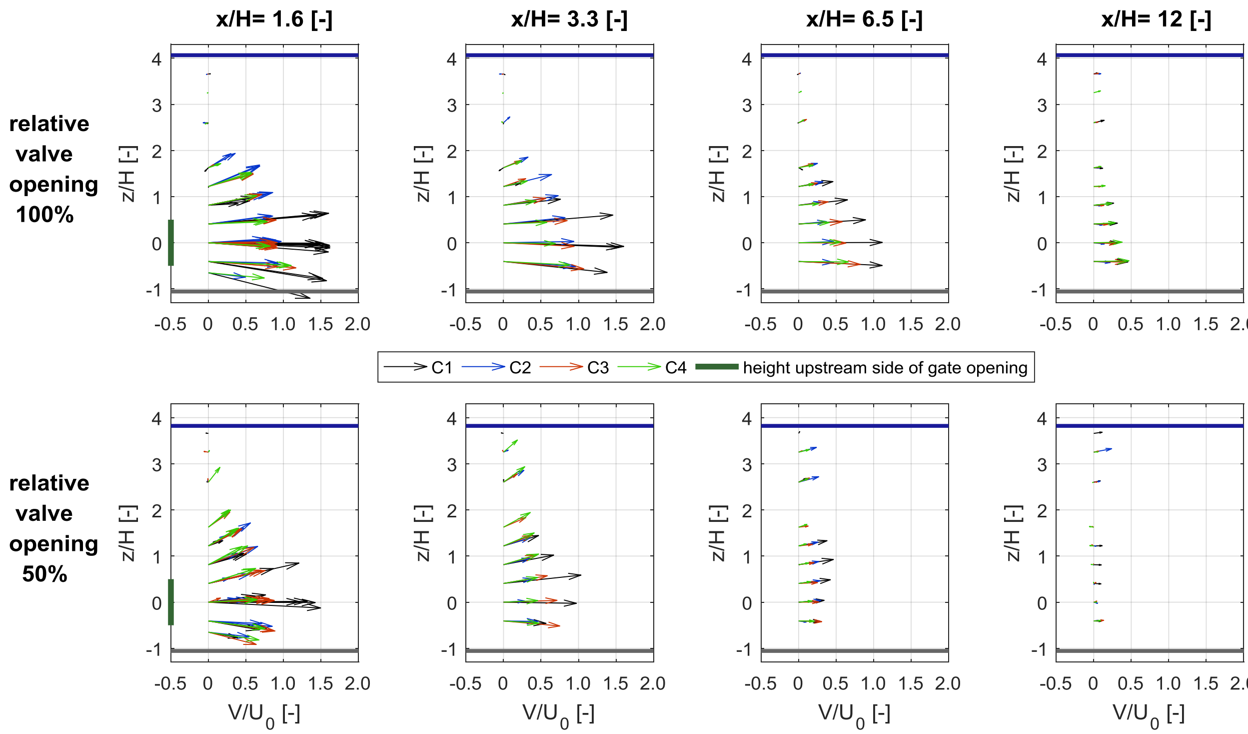


Figure 10: Vertical velocity patterns in the model's symmetry plane (y/W = 0)

Figure 9 and Figure 10 show that for the configuration without breaking logs (C1) the highest velocities are measured in the center of the gate opening. For locations near the center of the gate opening, significantly lower velocities are measured. For the configuration with 7 breaking logs (C2) and the configuration with 5 breaking logs (C3) the spreading of the jet in the vertical direction increases both for the fully opened and the half opened valve. The increased spreading of the jet in the horizontal direction is limited compared to the configuration without breaking logs. For the configuration with 3 breaking logs (C4), there is a difference in the flow pattern for the fully opened and the half opened valve. For the fully opened valve, the spreading of the jet in the horizontal direction increases and at x/H=1.6 three regions with higher velocities can be noticed at the centerline of the opening (z/H =0.0). For the half opened valve, however, the velocity pattern for the configuration with 3 breaking logs (C4) is similar to the velocity patterns of the other configurations with breaking logs. The conclusions on the spreading in vertical direction based on the dye visualizations (section 5) are retrieved in the results of the velocity measurements. For the configuration with 7 breaking logs (C2) and the fully opened valve there is an increased spreading in vertical direction towards the water surface, compared to the other two geometries with breaking logs. For the half opened valve, the spreading in vertical direction towards the water surface is the highest for the configuration with 3 breaking logs (C4).

The decay of the velocity in the lock chamber along the centerline of the opening (i.e. the x-axis) is presented in Figure 11.

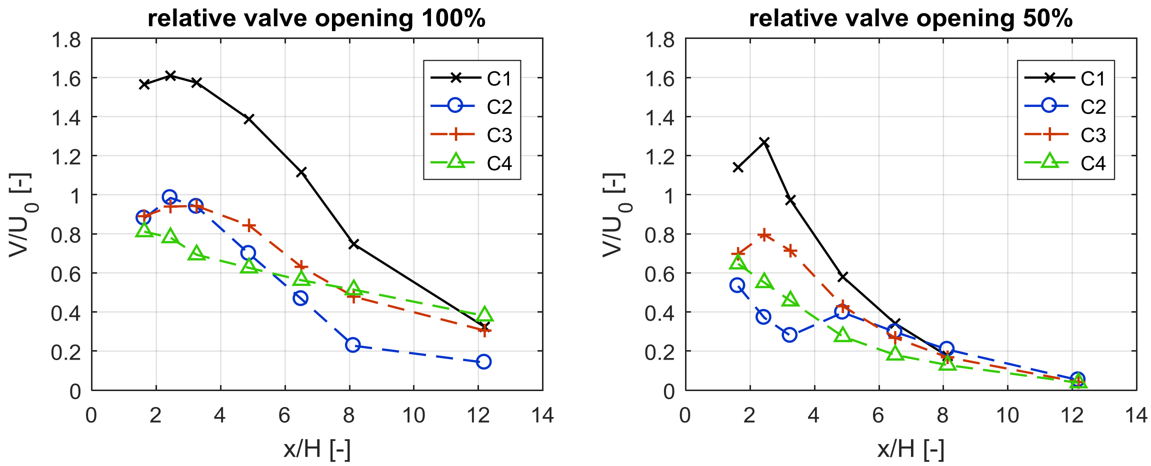


Figure 11: Velocity decay along the centerline of the opening (y=0 and z=0)

For the configuration without breaking logs and the fully opened valve, the maximum dimensionless velocity of 1.6 is measured at x/H=2.45. Note that the value of 1.6 corresponds well to the theoretical value of 1.56 (=1/0.64) derived above. Beyond x/H=2.45 the velocity on the centerline shows a continuous decay with an increase in distance. At x/H=12 the dimensionless velocity is reduced to 0.3. For the measurements without breaking logs and the half opened valve, a maximum dimensionless velocity of 1.3 is measured at x/H = 2.45. Note that the value of 1.3 is somewhat lower than the theoretical value of 1.43 (=1/0.69). It should be emphasized that for the half opened valve, the velocities are also measured at the centerline of the fully-opened opening. Due to contraction of the flow it is to be expected that for the measurement with the half opened valve the velocity is measured above the core of the jet. This is a plausible explanation for the velocity increase from x/H = 1.6 to x/H = 2.45. The decay in velocity with an increase in distance is higher for the measurements with the half opened valve compared to the measurements with the fully opened valve. This can be, partially, explained by the use of the full lift height H to scale the distance in x-direction both for the fully opened and the half opened valve. Adding breaking logs reduces the velocity. For the other configurations, the velocity on the centerline of the opening, for the measurements with the fully opened valve, is reduced with approx. 40 % (=1-1.0/1.6) for the configuration with 7 breaking logs (C2) and for the configuration with 5 breakings logs (C3). For the configuration with 3 breaking logs (C4), a reduction of approx. 50 % (=1-0.8/1.6) is observed. Note that in Figure 9 and Figure 10 for the configuration with 5 breaking logs (C3) and for the configuration with 3 breaking logs (C4) somewhat higher velocities are measured at z/H=-0.41 and y/W = 0.00, i.e. a dimensionless velocity 1.2 for the configuration with 5 breakings logs (C3) and a value of 1.1 for the configuration with 3 breaking logs (C4). At this point, the reduction of the velocity due to the breaking logs equals 25 % for the configuration with 5 breaking logs (C3) and 31 % for the configuration with 3 breaking logs (C4). For the configuration with 7 breaking logs (C2) and the configuration with 5 breakings logs (C3) and the fully opened valve, the velocity increases from x/H = 1.6 to x/H = 2.45. Beyond this point there is a continuous decay till the last velocity measurement at x/H = 12. For the configuration with 3 breaking logs (C4) and the fully opened valve, there is a continuous decay till the last velocity measurement at x/H = 12. At x/H =12 the velocity for the configuration with 5 breakings logs (C3) and the configuration with 3 breaking logs (C4) is in the same range of the velocity for the configuration without breaking logs, while for the configuration with 7 breaking logs the velocity is about half. The reduction of the velocity for the configuration with 7 breaking logs (C2) is most probably linked to the increase in vertical spreading, as observed both in the dye visualizations and the velocity patterns. For the measurements with breaking logs and the half open valve, the configuration with three breaking logs shows a continuous decay of the velocity, while for the configuration with 7 breaking logs (C2) an increase in velocity is noticed from x/H = 3.3 to x/H = 4.8 and for the configuration with 5 breaking logs (C3) an increase in velocity is noticed from x/H =1.6 to x/H =2.45. At x/H =12, the velocity for the configuration with breaking logs and the half opened valve ranges from 0.03 to 0.04.

7. CONCLUSIONS

For a lock leveling system with openings through the gates, mounting of breaking logs (also referred to as energy dissipation bars) at the downstream side of the gate openings is recommended to reduce the forces on the ships moored in the lock chamber. Since the shaping of a gate opening across the thickness of a steel gate and the integration of the valves are somewhat country-specific, it was decided to set up a generic physical model at Flanders Hydraulics Research (Antwerp, Belgium) aiming at determining the effect of breaking logs on the flow inside the lock chamber and optimization of the breaking log configurations adopted in Flanders (Verelst *et al.*, 2016). In this paper an account is given of the specific research carried out in this physical model for the rectangular gate openings of the new lock of Sint-Baafs-Vijve on the river Lys.

Four different configurations have been tested: one without breaking logs (denoted C1) and three configurations with breaking logs (denoted C2 to C4). The configurations with breaking logs differ in number of logs, spacing between the logs and (as far as C4 is concerned) width of the logs and positioning with respect to the upstream gate opening.

First, the discharge coefficients of the configurations were determined for relative valve openings ranging between 20 % and 100 %. It turned out that the influence of the breaking logs on the discharge coefficient was negligible for relative valve openings lower than 50 % and limited for higher relative valve openings.

Secondly, the effect of breaking logs on the spreading of a filling jet was studied. To this end, the velocity fields associated to a filling jet were measured by means of a dense grid of pointwise velocities, using an ADV device attached to an automated traverse system. In addition to the velocity measurements, flow patterns in the lock chamber were also visualized by means of dye injection. When adding breaking logs, the spreading of the filling jet in the vertical direction increased and the maximum velocity on the centerline of the inlet opening is reduced with 40 to 50 %, depending on the geometry of the breaking logs. The lowest velocities were achieved with the configuration with the least blockage of the gate opening at the downstream skin plate, i.e. the configuration with three breaking logs. The research revealed that the exact positioning of the breaking logs is more important for the spreading of the filling jets than the amount of blockage of the opening.

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DEVELOPMENTS IN RADIO NAVIGATION SYSTEMS

by

Michael Hoppe[1], Rainer Strenge[1]

ABSTRACT

Position, Navigation, and Timing (PNT) is part of the critical infrastructure necessary for the safety and efficiency of vessel movements, especially in congested areas such as the North Sea. GNSS (especially GPS) has become the primary PNT source for maritime and inland waterways navigation. The GNSS position is used both for vessel navigation and as the position source for AIS.

Furthermore the IMO e-Navigation concept supports the development of resilient positioning, navigation and timing (PNT) information. Further it is acknowledged that a number of technically dissimilar systems are required to ensure resilient PNT. The combined use of PVT relevant sensors (e.g. GNSS Receiver, DGNSS corrections, Multi-Radionavigation Receiver) and on-board systems (e.g. Radar, Gyro, etc.) establishes the needed redundancy to enable the monitoring of data and system integrity and to improve the performance of provided PNT data. This enables the protection of the on-board process of PNT data generation (cybersecurity) against intrusions by malicious actors.

Unfortunately, GNSS is vulnerable to jamming and interference, intentional or not, which can lead to the loss of positioning information or, even worse, to incorrect positioning information. One potential source of resilient PNT services is the use of Ranging Mode (R-Mode) using signals independent of GNSS. The concept of R-Mode, or ranging mode, was first introduced to the IALA ENAV Committee many years ago. It is a novel way of using existing maritime radio systems (MF radio beacon as well as AIS) to provide GNSS independent PNT. R-Mode could support resilient PNT by providing terrestrial positioning in coastal waters. First developments of this system concept were conducted in a feasibility study as well as a practical field demonstration within a transnational EU project named ACCSEAS (Accessibility for Shipping, Efficiency, Advantages and Sustainability) which ended in February 2015. A follow up project (R-Mode Baltic) has just started (10/2017) to provide a large transnational testbed for dynamic tests and to further develop the R-Mode technology.

The paper presents a brief description of the present developments in the field of radio navigation systems to be used in maritime and inland waterways. These systems comprise usable GNSS constellations, terrestrial radio navigation systems (like R-Mode) as well as the integrating aspect to provide resilient PNT for safety related applications.

1. INTRODUCTION

A reliable knowledge of a ship's position and movement in relation to other traffic participants and obstacles is a fundamental requirement for navigation and to avoid collisions and groundings. This holds true for maritime navigation as well as for shipping on inland waterways. Position, Navigation, and Timing (PNT) is part of the critical infrastructure necessary for the safety and efficiency of vessel movements, especially in congested areas such as traffic separation areas, harbour entrances or busy inland waterways.

GNSS (especially GPS) has become the primary PNT source for maritime and inland waterways navigation. The GNSS position is used both for vessel navigation and as the position source for AIS. Thus the dependency on an electronic position (as provided typically from a single GPS sensor) has increased over the last years (see **Figure 1**).

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Furthermore the IMO e-Navigation concept supports the development of resilient positioning, navigation and timing (PNT) information for maritime vessels. Further it is acknowledged that a number of technically dissimilar systems are required to ensure resilient PNT. The combined use of PVT relevant sensors (e.g. GNSS Receiver, DGNSS corrections, Multi-Radionavigation Receiver) and on-board systems (e.g. Radar, Gyro, etc.) establishes the needed redundancy to enable the monitoring of data and system integrity and to improve the performance of provided PNT data. This enables the protection of the maritime on-board process of PNT data generation (cybersecurity) against intrusions by malicious actors.

On inland waterways we could observe new developments towards the use of driver assistance systems, like automatic track control or bridge warning systems. Such systems are also using GNSS and DGNSS to provide the required positioning accuracy and will ask for the same level of resilience, integrity and reliability as in the maritime field.

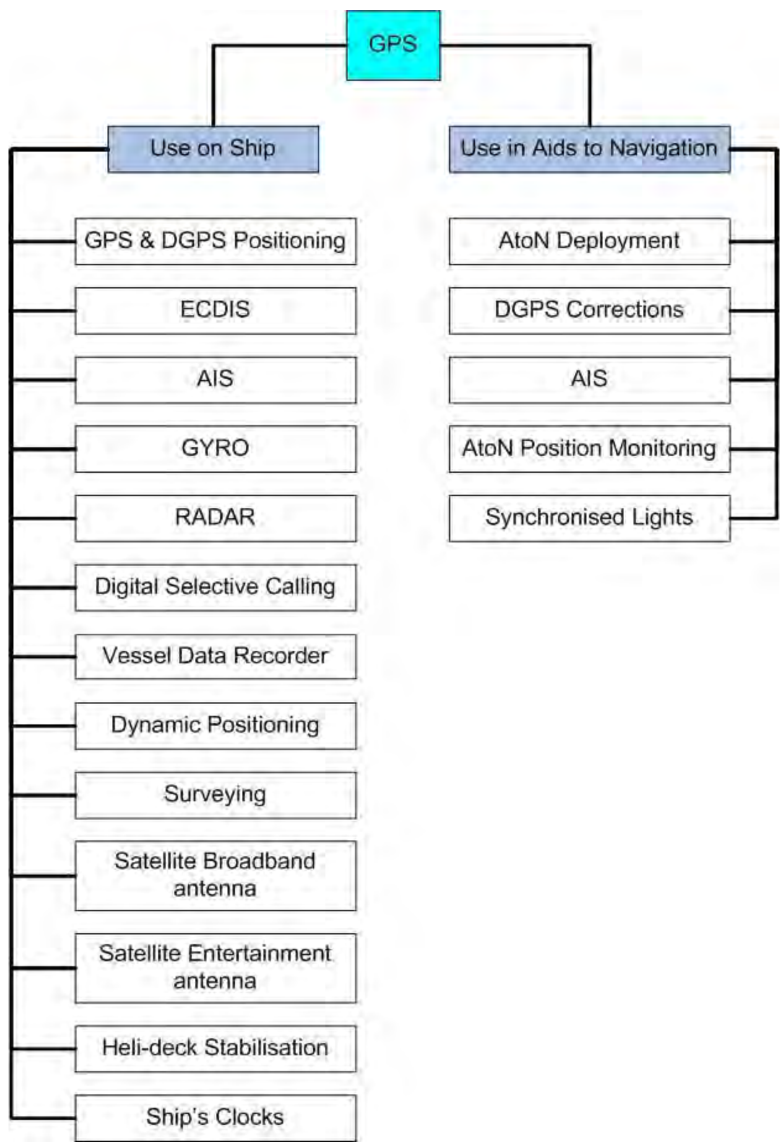

Figure 1 Dependency of an electronic position onboard a vessel

2. Systems and services used for maritime navigation

Nowadays the provision of electronic positioning is typically based on the US Global Positioning System (GPS). This system allows the usage of 32 global satellites which enable the use of the civil L1 frequency. After the deactivation of the selective availability in year 2000, GPS can be used with a positioning accuracy of roughly 10 m (DoD 2008). Also Russia is operating a satellite positioning system called GLONASS. Further systems are still under construction (European Galileo and the Chinese System Beidou) and will become operational within the next few years. Thus a lot of civil navigation signals will be available in near future to further improve the availability of GNSS positioning signals. Due to the fact that the existing and emerging GNSS could not fulfil all demands with respect to accuracy, continuity and especially integrity, coastal administrations are operating so called differential GNSS (DGNSS) to improve position accuracy and system integrity along their coastal waters. The use of GNSS independent terrestrial radio navigation systems is also discussed in the maritime field, due to the vulnerability of GNSS against intended (jamming and spoofing) or unintended (e.g. solar activities) interference. Here a system concept, called R-Mode, is under development which may become a backup in coastal waters to provide resilient PNT on board maritime vessels. The R-mode system concept will be further discussed in chapter 2.3.

2.1 Type of services, sensors and sources for positioning

Maritime positioning services could be classified as follows (IMO 2016):

- Radionavigation services provide navigation signals and data, which enable the determination of ships position, velocity and time (PVT data, as typically provided from GNSS).
- Augmentation services provide additional correction and/or integrity data to enable improvement of radionavigation based determination of ships position, velocity and time (e.g. DGNSS).

Furthermore such services could be classified regarding its geographical coverage:

- Global services are characterized by their world-wide coverage. They may have limitations regarding usability for different phases of navigation due to signal disturbances reducing the availability or performance of transmitted signals and/or provided data.
- Regional services (and maybe local services) are only available in dedicated service areas. They may be used to improve the performance of ships' navigational data in terms of accuracy, integrity, continuity and availability even in demanding operations.

Typically PNT data are provided from type approved sensors and data sources. They could be distinguished into the following categories:

- Service dependent sensors rely on any service from outside the ship provided by human effort. They cannot be used on board without at least a satellite-based or terrestrial communication link to the service provider (shown in **Figure 2**, mainly used to provide data of ships position, velocity and time).
- Shipborne sensors and sources:

  Primary sensors use a physical principle, e.g. earth rotation or water characteristics and are independent of any human applied service provision (shown in **Figure 2**, mainly used to provide data of ships attitude and movement);

  Secondary sensors and sources may be used to provide additional data for the verification of PNT data, e.g. water depth at known position from an ENC, line of position, or directions and distances provided by on-board RADAR.

The above described sensors are considered to be usable world-wide and free of any rebilling user charge.

In addition to sensors, services and sources listed in A.1 and A.2 further PNT-relevant data may be used for shipborne PNT data provision to increase redundancy or to evaluate plausibility and consistency of data input (ship sensed position e.g. by position reference systems). Such data may be provided via AIS or VHF Data Exchange System (VDES), see **Figure 2**.

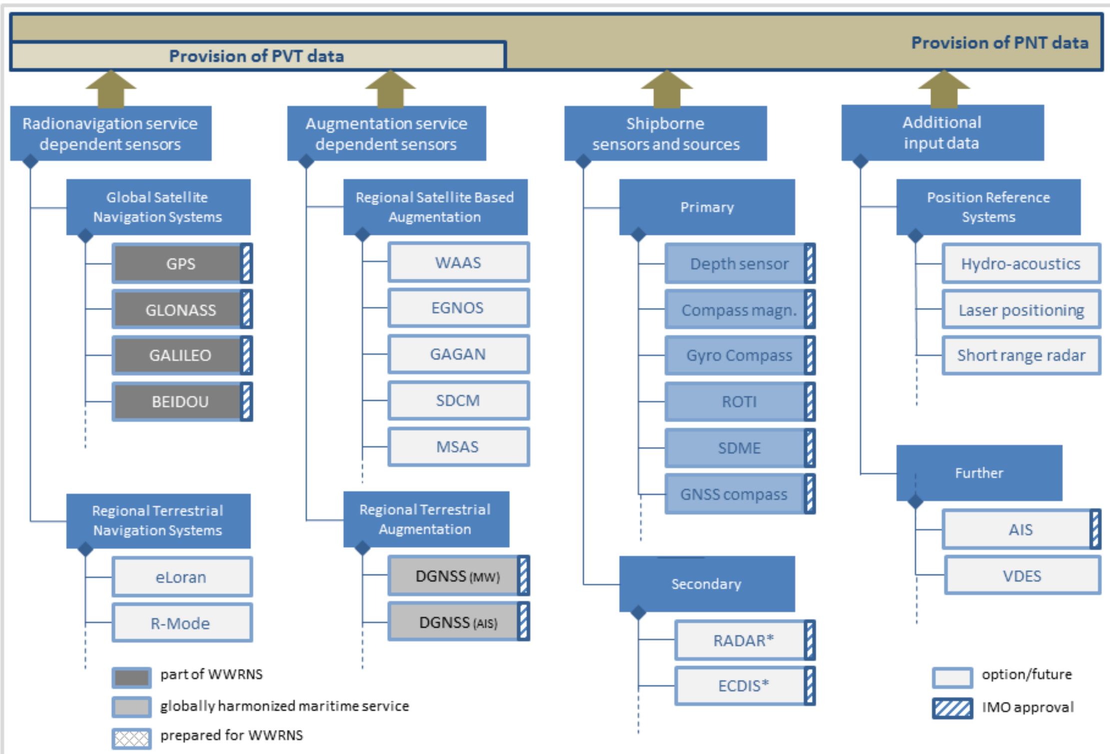

Figure 2: ***Systems, Services and Sensors in the maritime field, (IMO 2017)***

2.2 Requirements for Maritime Navigtaion

The International Maritime Organization (IMO) has defined operational requirements for the service provider of radio navigation (RNAV) services. To be recognized as part of the world wide radio navigation system (WWRNS), maritime RNAV systems have to fulfill the operational requirements of IMO Resolution A.1046(27), (IMO 2011). Within this resolution two different areas were defined with different sets of requirements (See **Table 1**).

Navigation Area	Absolute horizontal Accuracy	Signal Availability	Continuity	Integrity warning of system malfunction	Position Update Rate
NAVIGATION IN OCEAN WATERS	≤100m (95%)	>99.8%	-	as soon as practicable	2 s
NAVIGATION IN HARBOUR ENTRANCES, HARBOUR APPROACHES AND COASTAL WATERS	≤10 m (95%)	>99.8% (2 Jahre)	≥99.97% (15 Minuten)	< 10 s	2 s

Table 1: Operational requirements for RNAV systems being part of IMO WWRNS (IMO 2011)

In addition IMO has defined minimum maritime requirements for general navigation (IMO 2001).This document provide as well requirements for position accuracy, integrity, availability, continuity and position update rate for various navigation phases including inland waterways. Unfortunately no values are given for other navigational parameters (e.g. heading, COG, SOG, etc.). On the other hand the resilient onboard provision of position, navigation, and time data (**PNT**) is emphasized by the IMO e-navigation strategy, solution S3 “Improved reliability, resilience and integrity of bridge equipment and navigation information” and assigned risk control option RCO5 “Improved reliability and resilience of onboard PNT systems” (IMO 2008).

A comprehensive specification of maritime requirements on PNT data provision and integrity monitoring is a complex task. Many factors should be taken into account: ship types and carriage requirements, diversity of nautical applications and tasks, changing complexity of situation, deviations from nominal conditions up to customized level of support. Therefore it is difficult to determine the true development needs on the maritime PNT system regarding architecture, components, and functions to ensure a demanddriven provision of PNT data and associated integrity information. It should also be noted that during ship’s berth-to-berth navigation the requirements on data output of onboard PNT (data processing) unit vary in time and space as a result of changing environmental conditions and nautical tasks. The challenge for the maritime community is to find an efficient way specifying current and evolving requirements on PNT data provision.

An initial step towards resilient PNT has been realized by the maritime community with the development of the Performance Standards for multi-system shipborne radionavigation receiver equipment (MRR). This MRR PS supports the full use of data coming from current and future radionavigation systems and services. Consequently, the combined use of several GNSS and the additional use of Space Based Augmentation Systems (SBAS) as well as optional terrestrial radionavigation systems (e.g. eLoran or RMode) will be supported to increase the performance of positioning and timing.

As a second step the development of Guidelines for onboard PNT (data processing) Unit has been identified as supplementary and necessary. Initial point is the on board use of GNSS receivers (Global Navigation Satellite System) and autarkic systems (e.g. radar, gyro, echosounder with bathymetric data) in combination for a comprehensive provision of required PNT data. Redundancy in available data enables the application of integrity monitoring functions to evaluate the current usability of safety-critical data and components. Aim of the guidelines is the specification of data processing rules towards resilient provision of standardized PNT data and integrity information. For this purpose a modular architecture of onboard PNT system is introduced and scaled to the need on data input as well as the performance of data output.

2.3 R-Mode as a potential maritime Backup to GNSS

The need for resilient positioning, navigation and time (PNT) data has been well documented (Porathe 2013). Systems such as AIS (Automatic Identification System), ECDIS (Electronic Chart Display and Information System), ARPA (Automatic Radar Plotting Aid) and other navigation sensors use GNSS derived PNT data, the reception of which can be denied through natural and man-made interference. In the near future further GNSS will become fully operational (Galileo and BeiDou) which will further increase the number of available satellite signals. However, these all GNSS share similar signal structures, frequency bands and low signal power levels, and therefore have a common vulnerability to signal interference (Volpe 2011). Thus the development of an alternative backup system is recommended.

R-Mode (Ranging Mode) is a proposed terrestrial backup navigation system, independent to GNSS, which uses ranging signals typically transmitted from existing maritime infrastructure, for example, medium frequency (MF) radio beacons and/or AIS base stations, with the potential to be a component of the future VHF Data Exchange System (VDES).

Adding additional R-Mode functionality to existing maritime infrastructure is appealing, as much of the hardware is already in place, removing the need to procure and install expensive transmitters and antenna systems. In addition, the broadcast frequencies are protected and already established for this maritime radionavigation use and marine radiobeacon reference stations are already installed along most major shipping routes. AIS base stations have also been installed in significant numbers around many coastlines, have protected frequencies and already serve the mariner; and therefore are a good candidate for R-Mode transmissions.

The concept of R-Mode has been developed over a number of years and through various funded and national interest projects, each of which is summarized below.

2.3.1 Early Considerations

First ideas of an R-Mode system, which make use of existing maritime radio systems, were given as an input paper to IALA eNavigation committee in 2008 (Oltmann et al 2008). The main idea was to add a synchronized timing signal to existing radio infrastructure used in the maritime domain and has a worldwide distribution. Identified systems in this respect were the MF radio beacon system which provides GNSS corrections in the radio beacon band (at 300 kHz) and the AIS shore systems transmitting in the VHF band). Although the general idea of R-Mode was presented on several conferences no detailed investigation could be performed. A first detailed investigation on potential methods and an analysis of achieved R-mode performance of could be performed in the European ACCSEAS project.

2.3.2 The ACCSEAS-Project

The European collaborative project ACCSEAS developed the idea of R-Mode by supporting a feasibility study which considered the suitability of adding ranging information to marine radiobeacon DGNSS and AIS base stations.

The ACCSEAS feasibility study was split into the following parts:

- Parts 1 and 2 examined the R-Mode potential of the MF DGNSS signal (Johnson et al 2014); the recommended approach was to add CW signals to the broadcast and to develop the pseudorange from the carrier phase.
- Part 3 and 4 examined the R-Mode potential of the AIS signal (Johnson et al 2014); the recommended approach was to estimate the pseudorange from timing bit transitions and requires no modification to the signal structure.
- Part 5 examined the combination of MF transmission together with AIS and the combination with eLoran which at that provided time operated from 5 stations around the North Sea area (Johnson et al 2014).

The performance assessments for each of the three signal types outlined above were considered in the feasibility study, and the lower bounds of the expected positioning accuracy were calculated, based on conservative assessments.

As the position is calculated through trilateration from terrestrial transmitters, the resulting performance is a function of the received signal power, the observation time of the receiver (nominally assumed to be 5 seconds), and the geometry of the known transmitter locations. For each signal considered, sources of error were considered where possible, however errors such as unknown offsets in the synchronization of transmitters (this is relevant to all three signals) and propagation delays that would increase the observed range estimates (this is known to impact both MF and eLoran signals since they propagate as ground waves) were omitted.

The ACCSESS ACCSEAS project concluded in 2015, and the project information remains available on the project website [www.accseas.eu] with the project deliverable reports available on the IALA website, within the e-Navigation test bed area.

2.3.3 R-Mode via MW-DGNSS

The ACCSEAS feasibility study considered a number of possible methods of adding a ranging signal to the existing marine beacon system. The different options considered are outlined in the ACCSEAS feasibility study (Johnson et al 2014), from which the approach of adding two CW signals in the MSK spectrum was selected as the optimum solution at this time. The selected approach was to maintain the existing MSK signal for legacy users, but to add two continuous wave transmissions to the band, one above and one below the central frequency. The time of arrival (TOA) of the R-Mode signal will be determined by using phase estimation methodology. Due to a wave length of roughly one km there is a need to solve the ambiguity within the nominal range of a radio beacon (typically < 300 km). The addition of a separated CW signal allows for the ambiguity solution (using the beat frequency between the added CW's).The legacy MSK signal may be used to provide other information to the receiver. Further the data stream used for DGNSS transmissions may be another alternative to resolve the ambiguity. The ACCSEAS report suggests the two CW signals are positioned ±250Hz with respect to the center frequency, however this has been amended to ±225Hz to prevent overlap of CW signals between neighboring (in terms of frequency) stations. The ACCSEAS feasibility study provides an estimation of the expected accuracies of marine radiobeacon R-Mode for day and night time conditions, taking into account the radiobeacon sites located in the North Sea region (**Figure 3**). The theoretical performance by day is promising with the expected accuracies in the order of 20-30m deemed possible. At night the expected accuracy drops to 90+m as the effect of skywave interference is observed.

Further consideration and work is required to understand the impact of skywave effects and whether they can be suitably mitigated for MF R-Mode solutions. This question and others will need to be addressed in due course and will form part of the further work.

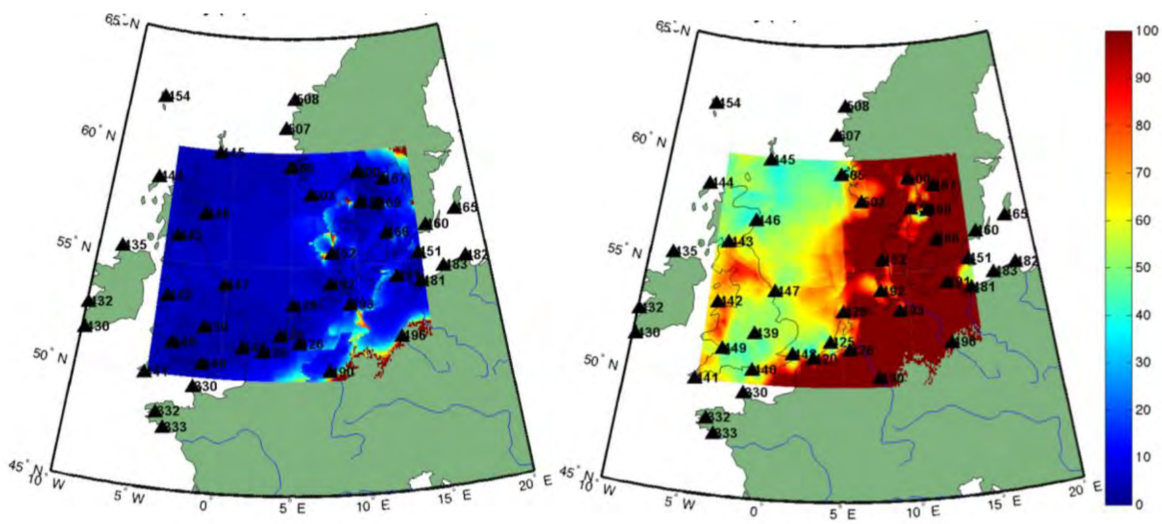

Figure 3: MF DGNSS R-Mode day (left) and night (right) predicted positioning accuracy (m) using a 0-100m scale (Johnson et al 2014).

2.3.4 R-Mode via Automatic Identification System (AIS)

AIS broadcasts are in the Very High Frequency (VHF) maritime mobile band (156.025 - 162.025 MHz) and currently transmit information both ship-to-shore and shore-to-ship (the base station broadcasts) using Gaussian minimum shift keying (GMSK) in a time division multiple access (TDMA) mode. Within part 3 of the ACCSEAS feasibility study (Johnson et al 2014) the following potential ideas and methods to implement VHF R-Mode were identified.

1. Existing AIS: this solution involves ranging off of the existing base station AIS messages, using Message 8s to increase the signal energy and duty cycle.
2. CW Aiding: this solution consists of adding continuous wave (CW) signals in other VHF channels and ranging off of the carrier phase of beat signals generated from pairs of such CW signals.
3. Spread Spectrum: this solution considers using more of the VHF bandwidth by transmitting direct sequence spread spectrum signals, akin to GNSS pseudolites.

For the Existing AIS case, the analysis shows that the TOA performance is a function of the number of bits in the processed message(s) and the signal energy and that the performance ranges from 225m for a marginal signal level to 5m for a typical signal level. The raw TOA performance of CW Aiding using the CW carrier is a function of signal strength, averaging time, and frequency and can achieve sub-meter accuracy. However, since the cycle ambiguity must be resolved, the overall performance depends upon how many, and what frequency, CW signals are added. For example, 10m ranging accuracy is possible using 3 CW signals. Finally, while a complete analysis of the Spread Spectrum solution is beyond the level of this report, we argue that performance at a level of approximately 12 meters appears achievable over a limited coverage area.

The recommended solution is the first, existing AIS including Message 8s. With this solution 10m performance appears achievable using the existing system with no modifications other than adding some additional transmissions. The CW solution is not preferred as it requires additional VHF channels plus adds the complexity of resolving cycle ambiguity. Spread spectrum, while interesting, is also not preferred as its coverage area is likely to be more limited than the other solutions.

The ranging performance is impacted by a variety of factors that are explored in this report: time stability and synchronization, signal power loss with distance, noise levels, and geometry. In the position analysis it is assumed that the time stability (on the order of 1 ns) and synchronization (to within 50-100ns) to a common reference such as UTC is achievable. Algorithms to predict power as a function of distance for the line-of-sight transmission of VHF signals are known. Noise in the VHF band has been previously studied. The geometry of the position solution, as measured by the Horizontal Dilution of Precision (HDOP), is a major factor in overall positioning performance, but HDOP values in the North Sea Area are quite good (generally less than 2).

The predicted bound on R-Mode positioning using TOA accuracy bounds is good (10-30 m) in most of the investigated area (North Sea Area, see **Figure 4**). Accuracy at the 10m level could be achieved in critical areas (port entrance) e.g. by adding additional transmitters to improve station geometry.

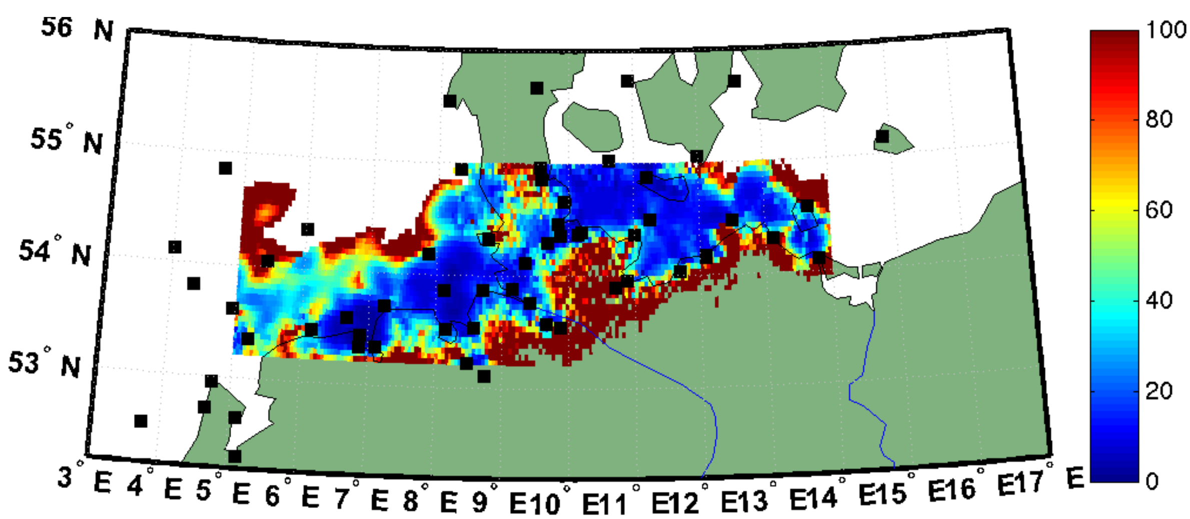

Figure 4: AIS R-Mode predicted positioning accuracy (m) using a 0-100m scale, (Johnson et al 2014)

2.3.5 Combination of various R-Mode-Signals

Furthermore the ACSSEAS study considered the combination of R-Mode signals for a better position solution (Johnson et al 2014). Part 3 of this study describes the architecture of an "all-in-view" receiver that could combine signals from two or more sources (e.g. MF, AIS and eLoran). The potential performance of this all-in-view receiver was computed for the North Sea Area with various combinations of the above mentioned signals to demonstrate potential synergies. As expected, the best performance can be achieved using ALL signals. Performance at night is slightly worse than during the day due to the reduced MF DGNSS performance at night. A single eLoran station can provide a large improvement at night over a large coverage area. Sylt alone for example, can cover the North and Baltic Seas sufficiently as an R-Mode signal to supplement AIS or MF DGNSS R-Mode; it is a very efficient augmentation to AIS R-Mode. Also, even a single strategically positioned eLoran station can provide cost-efficient time transfer for all R-Mode MF and AIS shore stations in such an area; precise timing of transmissions is one of the fundamental pre-requisites for any R-Mode at MF and/or AIS shore stations. In summary, depending upon availability, 1 or 2 eLoran signals can be combined with AIS and MF DGNSS to offer improved performance. In general performance results are strongly position dependent – in many areas one system (signal type) dominates performance. Also, as expected, more signals results in increased performance (or at least no worse). To achieve widespread resilient PNT, the best solution is to use all signals available in a true all-in-view receiver. It is also important to note that the need for a backup PNT is not uniform. The performance of a backup PNT system is most critical in the areas with the highest density of shipping traffic. **Figure 5** shows the predicted R-Mode performance for region II overlaid on top of the ACCSEAS shipping density plot to illustrate this alignment. It is likely that this alignment of R-Mode performance with the high density shipping lanes will be true in other parts of the world as well, since those are the areas with the largest numbers of AIS (and MF DGNSS) stations.

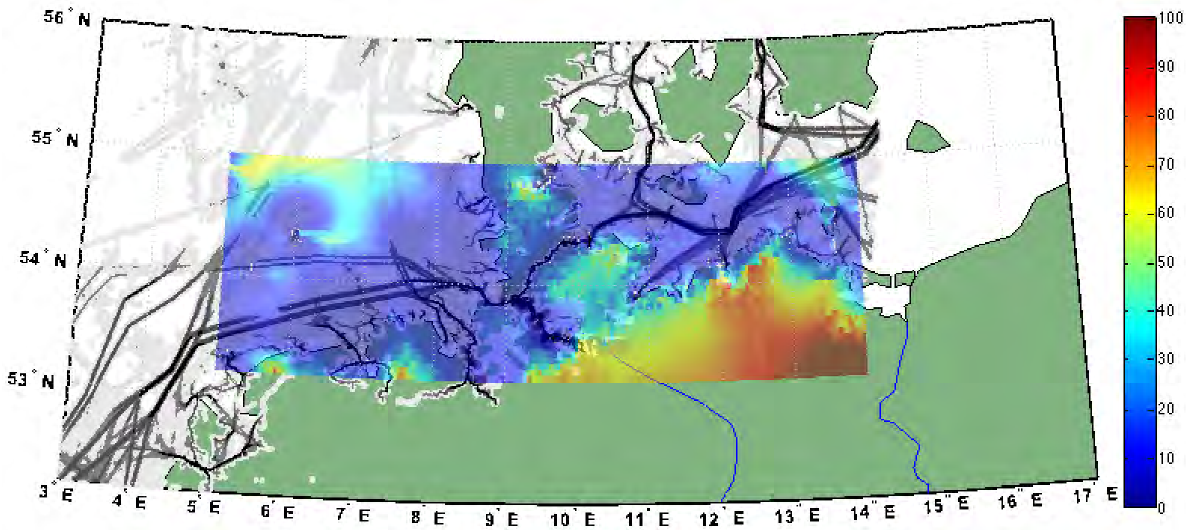

Figure 5: R-Mode performance overlaid on top of shipping density plot. Predicted R-Mode performance shaded using 0-100m accuracy scale. Shipping density shaded so that darker is higher density traffic, (Johnson et al 2014).

3. DEVELOPMENTS ON INLAND WATERWAYS

Considering the growing interest in a more efficient use of inland waterways a lot of new developments have been undertaken within the European RIS (River Information Services) concept with the following aims: economical use of the existing inland waterways improving the safety of the inland waterways increasing the commercial benefit for the skipper reducing the workload of the skipper One major improvement is the development of an Inland ECDIS (Electronic Chart Display and Information System) and the appropriate standard which is compatible to the existing maritime ECDIS. Also the development of a system for vessel tracking and tracing on inland waterways will lead to an important tool to exchange navigational data between ships and between ships and shore stations. One essential requirement for such new telematic applications is the use of an appropriate radio navigation system, like DGNSS. The use of DGNSS will provide a position with high accuracy, availability and integrity which will significantly enhance the safety for RIS applications (Hoppe 2016).

3.1 Present Applications and Requirements

2.3.6 Inland ECDIS

As mentioned before one major improvement is the development and introduction of an Inland ECDIS and the appropriate standard which is compatible to the existing maritime ECDIS. When using such a system in "navigation mode" the inland ECDIS provides information about the precise position of the ship in relation to the fairway edge. This development is based on a task providing Rhine river depths information on an electronic chart (Krajewski et al 2002).

Within the Inland ECDIS standard two modes of applications are described:

- information mode
- navigation mode

In the information mode the electronic chart is displayed without the overlaid radar picture. The skipper can use this mode to collect information about the waterway or for journey preparation. The Inland ECDIS information mode has low demands concerning position accuracy, availability and integrity. Hence a typical GNSS receiver could be used to position the map, e.g. on a typical PC. **Figure 6 (left)** shows an inland ECDIS chart in information mode. It should be mentioned that it is not allowed to use the information mode for navigation purposes. When using the Inland ECDIS in navigation mode the radar picture is overlaid the inland chart. Near the typical information about the waterway the chart will also contain depth information with respect to the current water level, the draught of the ship and the required safety margin. Equipped with all this information the skipper is able to maximise the cargo during a journey. As a result of increasing the draught of the ship the map will show the reduced fairway in parts of the waterway. It is obvious that using the Inland ECDIS in navigation mode will effort a positioning system with high demands concerning accuracy, availability, continuity and integrity. **Figure 6 (right)** illustrates the Inland ECDIS in navigation mode.

Figure 6: Inland ECDIS in information mode (left) and navigation modus (right)

2.3.7 Inland AIS

The Automatic Identification System (AIS) is a ship-borne radio data system, exchanging static, dynamic and voyage related vessel data between equipped vessels and between equipped vessels and shore stations. Ship borne AIS stations broadcast the vessel's identity, position and other data in regular intervals. By receiving these transmissions, ship borne or shore based AIS stations within the radio range can automatically locate, identify and track AIS equipped vessels on an appropriate display like radar or Inland ECDIS (CCNR 2011). In inland navigation the European RIS platform, the Central Commission for Navigation of the River Rhine and the Danube Commission have considered AIS as a suitable technology that can also be used for automatic identification and vessel tracking and tracing in inland waterways. An inland AIS standard is developed to serve the specific requirements of inland navigation while preserving full compatibility with IMO's maritime AIS and already existing standards in inland navigation.

The German Federal Waterways and Shipping Administration (WSV) has installed an AIS network which covers the coast as well as major inland waterways like River Rhine, Moselle, Danube, Main and Main Danube channel. It is planned to expand this coverage within the next years to enable a nationwide AIS shore infrastructure along most inland waterways. Furthermore a mandatory carriage requirement already exists on the River Rhine and Moselle (CCNR 2015). Meanwhile the carriage requirements have been extended to all vessels on German inland waterways since beginning of 2017. The described applications have specific requirements with respect to position accuracy, availability, continuity and integrity which are detailed in the performance standards on vessel tracking and tracing (CCNR 2006) and inland ECDIS (CCNR 2003). **Table 2** provides an overview about these performance requirements.

Application	**Position accuracy [m]**	**Integrity**
Inland ECDIS		
Navigation Mode	< 5 (absolute) < 5 (1 σ)	Detection on errors > 3 σ within 30 seconds
Information Mode	-	-
Inland AIS		
- Medium-term ahead	15-100	-
- Short-term ahead	10	-
- Lock/Bridge operation	1	-

Table 2: Performance requirements for Inland AIS and Inland ECDIS

To provide the required position accuracy and integrity for inland AIS and Inland ECDIS in navigation mode the WSV has implemented a DGNSS service along German inland waterways in 2005 (Hoppe et al 2006). This service is based on the maritime radio beacon service according to the recommendation given by IALA (IALA 2015). The IALA DGNSS can provide accuracy in a range of 1-3 m and provide integrity information to the users in less than 10 seconds.

3.2 Future Developments

Beside the established applications (as described above) new driver assistance systems are studied in various research projects to further improve the safety of navigation and to develop the basic technologies for a future autonomous inland ship. One of this research projects is LAESSI (Control and Assistance Systems to Enhance the Safety of Navigation in Inland Waterways). The aim of the project LAESSI is to support the skipper in his tasks of guiding the vessel and thus make inland shipping safer and also more efficient. To reach this aim, LAESSI makes use of latest GNSS navigation technology and data transmission developments. Within the project LAESSI, which is funded by the Federal Ministry for Economic Affairs and Energy (Bundesministerium für Wirtschaft und Energie, BMWi), efficient navigation assistance functionalities for the inland waterway transport will be developed. The application of these novel functionalities aims at the enhancement of safety and efficiency of inland waterway transport by reducing the risk of collisions. The need for this is indicated by a number of recent accidents involving inland vessels. Specifically, the project focuses on the development of the following navigation assistance functionalities: bridge approach warning system, guidance assistance and mooring assistance including the associated Conning Display.

- The bridge approach warning system should provide a timely warning signal to the skipper, whenever the vessel and particularly the wheelhouse or the radar mast cannot safely pass under the bridge. A possible warning has to be issued several hundred meters before the bridge is reached in order to ensure a sufficient reaction time.
- For the mooring assistance the position and attitude of the vessel has to be linked to the surroundings of the vessel. The skipper should get an accurate representation of the actual situation, in particular, the current distances to piers and other vessels and will be supported by this information during the maneuvers.
- The guidance assistance will relax the load on the skipper during the on route navigation of the ship. A highly accurate and integrity tested positioning information serves as a basis for this functionality.
- The Conning Display will present the motion of the ship in a clear form. Especially, the skipper has to be promptly informed about the relevant changes of the motion. For this purpose it is not only necessary to provide a very accurate position and attitude information of the vessel, but it is also important to consider the information from the propulsion systems as well as the influence of the wind and current.

The essential basis for the aforementioned functionalities is the provision of reliable and comprehensive nautical information. This includes the position, attitude and movement of own vessel with the associated integrity information, exact and valid electronic charts (maps) as well as the information regarding the situation in navigational area (construction sites, accidents, water levels) and characteristics of the infrastructure (e.g., dimensions of the bridges and locks).

In the following table accuracy as well as integrity requirements are listed for the different assistance functions. The basic idea of a bridge collision warning is to compare the geodetic height of the vessel with the geodetic height of a bridge. This results in requirements for the height measurement. The algorithm will further be divided in two parts, a long distance part starting about 10 minutes before the bridge passage and a close up part in the last 2 minutes. The first lines in **Table 3** in column bridge collision warning refer to the close up part. As a warning in the close up part requires immediate action of the skipper, higher integrity requirements have to be taken into account there. Also the time to alarm is lower there. Position and heading are used as a basis for prediction the ships movement in the close up part high requirements for position and heading accuracy are necessary of for the mooring assistant. These are based on the requirement that each point on a 185 m convoy shall be known with 30 cm accuracy. The integrity risk is based on the assumption, that 1 non detected error within 3 years of normal operation can be tolerated. The time to alarm requirement takes into account, that on the one hand mooring is a critical operation. On the other hand the vessel is moving at low speed in this situation.

	Bridge collision warning	Automatic guidance	Mooring assistance	Conning display
Position accuracy [cm]	20	30	10	20
Height accuracy [cm]	10	not relevant	not relevant	not relevant
Heading accuracy [°]	0,3	0,17	0,07	0,1
Integrity risk	10 10^{-5} / 2 min 30 10^{-5} / 8 min	0,55 10^{-5} / 3 h	18 10^{-5} / 10 min	18 10^{-5} / 1 h
Time to alarm [s]	4 6	2	6	6

Table 3: Accuracy and integrity requirements of assistance functions (Sandler et al 2016)

Quite similar are the requirements for the conning display. Here the vessel operates in confined waters, but most times not close to walls or other vessels. Thus accuracy requirements are lower. Also the time of using a conning display is longer than a typical mooring situation. For automatic guidance of the vessel no especially high accuracy is required. As a difference to the other assistance functions, the skipper is not part of the control and action loop. His functions are monitoring of the system as well as tactical trajectory planning. Jumps due to errors in the position measurement might have immediate impact on the rudder commands. Thus the function has to be very reliable. Basis for the proposed integrity risk is the assumption, that within one year on 100 vessels only 1 case of not detected integrity problems may occur. Also a very short time to alarm is required. The required level of accuracy (dm – cm) cannot be achieved by currently used code based positioning techniques. Therefore phase based GNSS positioning (RTK) needs to be applied. Furthermore exact and valid electronic charts, as well as information regarding the actual situation in the navigational area (e.g. temporary restrictions at construction sites) are required.

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TRAFFIC MANAGEMENT, RELIABILITY AND ECONOMIC TRANSPORT ON THE INLAND WATERWAY DANUBE

by

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ABSTRACT

The analysis on goods transport in the Danube together with cost factors and infrastructure reliability revealed the strengths, weaknesses, risks and opportunities of the Inland Waterway Danube. According to the results waterway transport will stay competitive or may even regain market shares if continuous reliable fairway conditions with an available water depth of at least 2.5 m can be provided even in low-water periods. Improving transport logistics e.g. with fixed contracts using mixed modes of transport and optimal loading depending on actual conditions are necessary as well. For managing a dynamic river as transport infrastructure in a cost-efficient and environmentally friendly way viadonau has teamed up with Vienna University of Technology and Hoffmann Consulting in order to develop a holistic Waterway Asset Management System (WAMS). In a first phase from 2012 to 2015 the principal methodological availability approach and a dredging management have been developed (WAMS 1.0). In the second phase from 2016 to 2018 additional functionalities for sediment, waterway structures, and traffic management have been implemented (WAMS 2.0). The development and implementation in a software tool has been work in progress providing constant feedback between theoretical considerations and practical results as new functionalities become available. Thus, the WAMS software tool is becoming the central database providing viadonau with the means to move from empiric reactive maintenance approaches towards quantitative asset management strategies with fast semi-automated processing capabilities and pro-active maintenance in a user-friendly environment. The focus of this paper is traffic management connecting the physical availability and its optimization with an analysis of actual traffic flows and utilization of the vessel fleet in real time. To achieve this goal anonymized transponder data leaving only vessel type, position and draught loaded are imported for calculating encounters, traffic distributions and fairway utilization. Based on these data it is possible to generate traffic heatmaps and assess critical encounters in narrow sections at low water periods as a basis for aligning the fairway path and defined levels of service. Furthermore, the WAMS is capable of monitoring the progress of pro-active dredging measures allowing a fast implementation and communication of results to the transport industry. With historic and actual data from riverbed surveys, water levels and traffic it is already possible to calculate the availability of any defined level of service for any transport route on the Danube in Austria in a matter of minutes. The possible loading of any vessel type can also be derived from calibration curves linking utilization and static draught with dynamic squat depending on vessel speed and necessary underkeel clearance. With the Ministers of Transport on all riparian countries of the Danube endorsing a common Fairway Rehabilitation and Maintenance Master Plan in 2014 (FAIRway) the EU - project WAMOS will lead to one common database on fairway conditions of the entire Danube. Combining these information with traffic analysis capabilities will allow both efficient investments in waterway availability as well as competitive pricing and transport planning on 2.400 river kilometer on the entire Danube until 2020.

KEYWORDS: Danube, inland waterway, availability, asset & traffic management, traffic distribution, fairway alignment

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1. INTRODUCTION

Transport infrastructures as important assets of national economies develop in very long cycles due to their service lives and costs whereas transport of goods (and passengers) are closely related to economic development (Hoffmann, M. 2017). Figure 1 provides an overview on the waterways in Central Europe with the Danube as a central transport axis. Figure 2 (top) shows the cross-sectional traffic volumes in the Danube corridor at the borders of Austria. To the west at Passau the total volume grew from 19.5 million tons in 1990 to 62.4 million tons in 2016 (+8.5% p.a.) and to the east at Hainburg from 12.9 to 44.2 million tons (9.3% p.a.) proving an increasing importance of international transport. During this period, there have been huge shifts in modal split from 13.2% down to 6.2% (Danube), 42.1% down to 27.9% (rail), and from 44.7% up to 65.9% (road) to the West. To the East these shifts have been even more significant from 48.9% down to 15.3% (Danube), from 38.6% down to 31.2% (rail), and from 12.4% up to 53.4% (road). In summary, transport volumes on rail and Danube have been stable or declining slightly with the majority of growth being allocated to roads. Fig. 2 (bottom) shows transit, import, export and domestic transport volumes in Austria with the main Ports being Linz and Vienna. Fig. 3 provides an overview on the entire goods transport shares on 2.400 km Danube in all riparian countries from Germany to the Black Sea. Long distance bulk good transport with the main commodities being ores & metal waste (26.7%), agricultural & forestry products (19.0%) and petroleum products (14.6%) are characteristic for the Danube. In contrast, passenger transport is still increasing setting a record high of 1.23 million in 2016 in Austria (cruise ships, and day-trip vessels) with good prospects for further growth.

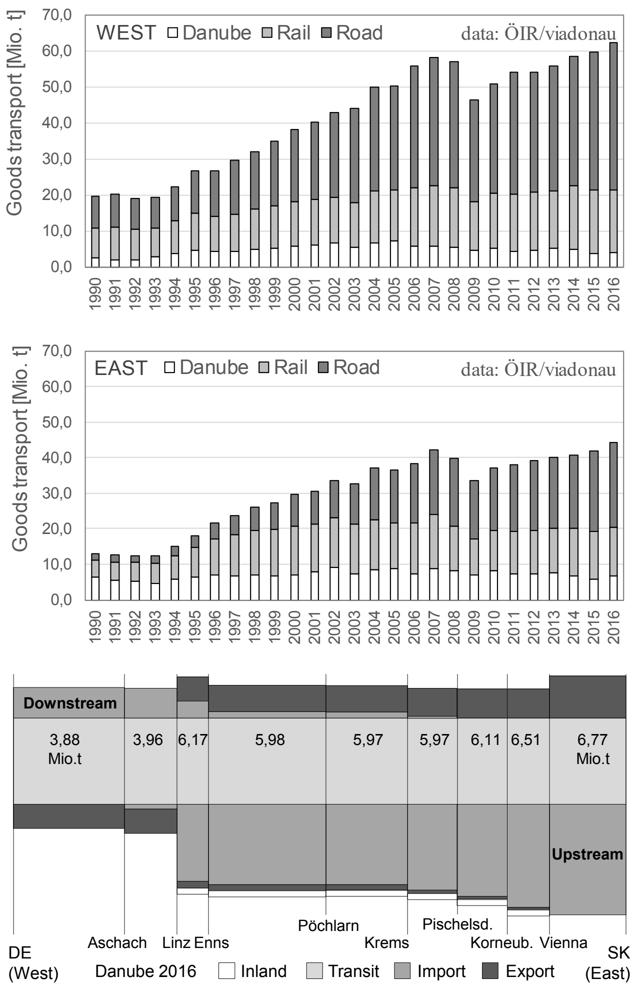


Figure 2: Traffic volume development in the Danube corridor in Austria from 1990 to 2016 and traffic flow analysis on the Danube in Austria 2016

Figure 1: Overview on Inland Waterways in Central Europe with the river Danube in Austria

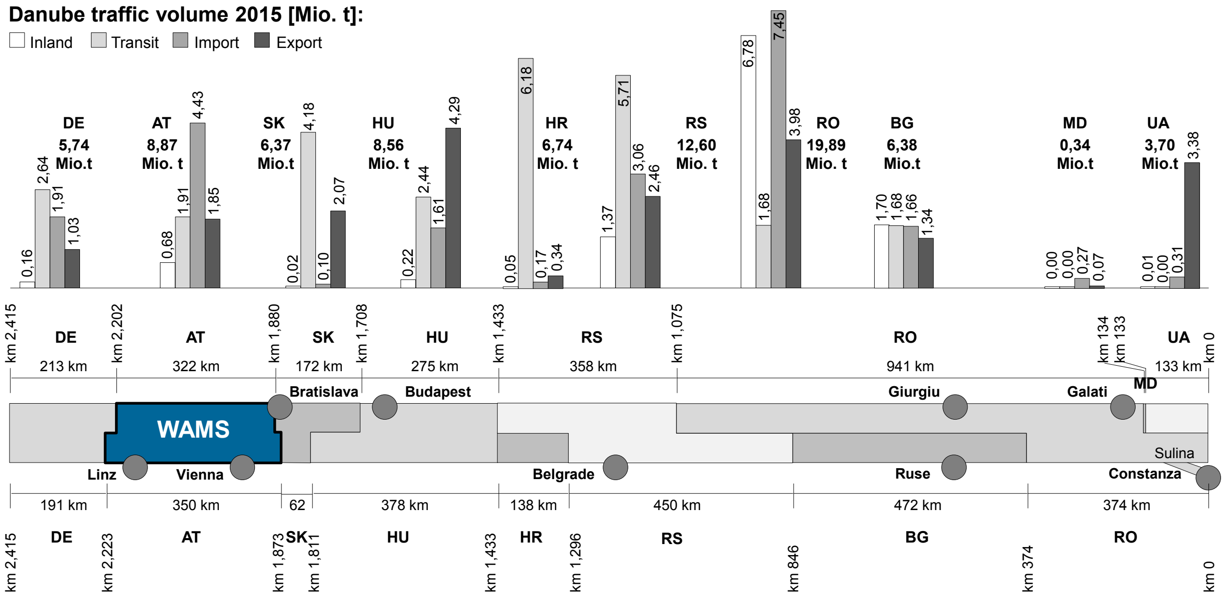


Figure 3: Overview total traffic volumes per country on the river Danube from Germany to Austria and the Black Sea in 2015 with Waterway Asset Management System WAMS in Austria

2. Waterway transport conditions in Austria

In general, the volume of regional transport markets decreases with increasing distance with the main decision factors in transport logistics of bulk goods being reliability and costs on the entire (multi-modal) transport chain (Hoffmann, M. et al. 2014). Fig. 4 shows the distance-related transport unit costs and modal dominance in the Danube corridor including pre- and end haulage in a catchment area of 50 km for road, rail and river Danube. In summary, transport on rail and inland waterways is economic on long transport distances at high loading factors due to large fixed and low variable cost components. Transport on waterways is especially sensitive as adverse weather conditions (ice, floods) may block vessels and low water depths are limiting utilization. With ongoing pressure on the transport market (e.g. overcapacities, low fuel prices) fairway depths between 2.0 to 2.4 m (average vessel utilization up to 50%) are insufficient as the resulting transport costs are far too high. However, if a high availability with reliable fairway depths above 2.5 m can be provided on entire transport routes waterway transport is becoming increasingly economic. Apart from transport cost analysis the proof may be found in the comparison of fairway availability and transport volumes in consecutive years. Due to an exceptionally warm and dry weather between July and December 2015 a water depth of 2.5 m east of Vienna was only available on 35% of this period with the total transport volume on the Danube in Austria dropping from 10.1 in 2014 to only 8.6 million tons in 2015. With better weather conditions and intense pro-active maintenance, the conditions on the Danube

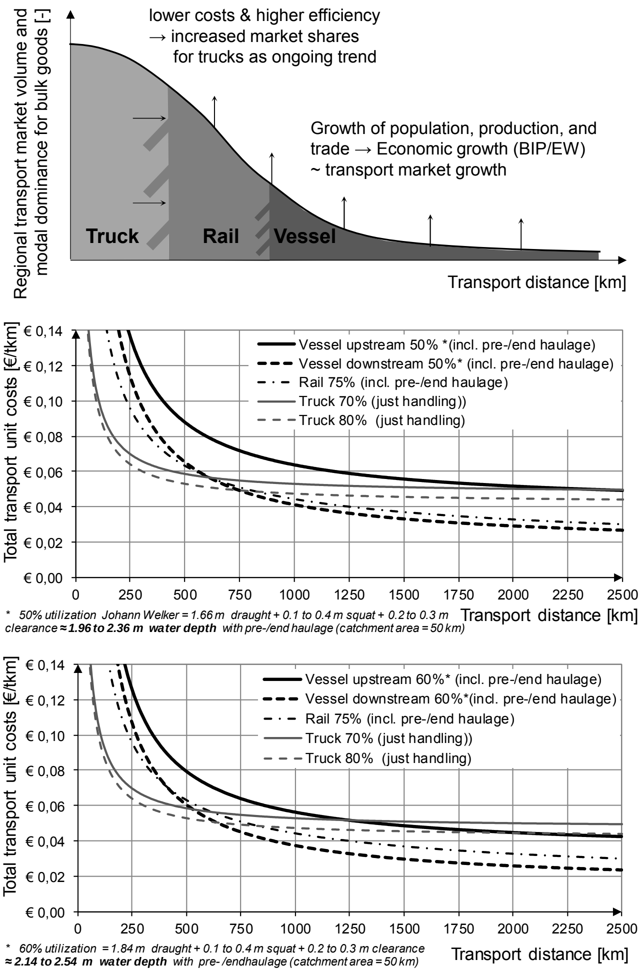


Figure 4: Distance related transport market (bulk goods) and transport unit costs of road, rail and waterway on the Danube Corridor 2016

have improved in 2016 (Fig. 5) with 89% availability downstream and 98% availability upstream. As a consequence, transport volumes on the Danube have recovered increasing to a total of 9.1 million tons at an average load factor of 61.7% (viadonau 2015/16).

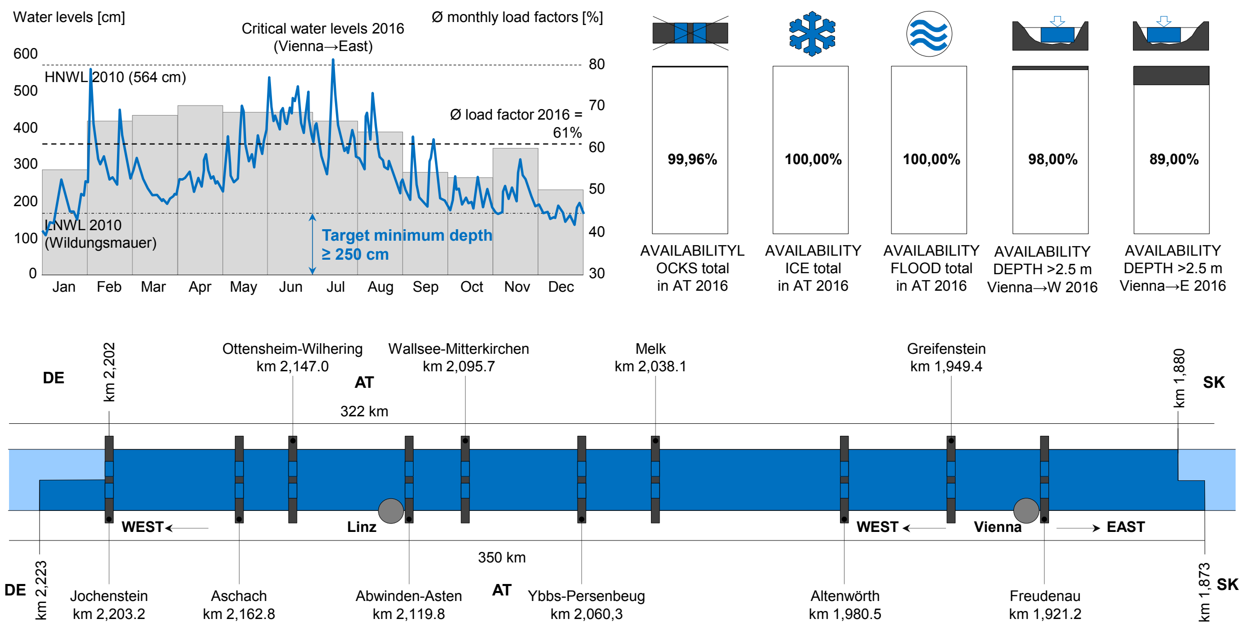


Figure 5: Key performance indicators waterway: Overview critical water levels, vessel load factors, locks and availability on the Danube in Austria 2016

3. Overview Waterway Asset Management System WAMS

Effective waterway management with sufficient fairway width and depth throughout the year is crucial for an economically successful development of waterway goods transport. Therefore, viadonau decided in 2012 to move from existing empiric reactive approaches towards an analytic proactive life cycle costing approach with the development of a waterway asset management system WAMS (Hoffmann, M. et. al. 2014a,b, Haselbauer, K. 2016). At the core of the developed approach is the total availability concept based on all possible available combinations of fairway width and depth in days per year with the levels of service accounting for typical encounter situations at a minimum fairway depth of 2.5 m (Fig. 6). The principal methodological approach and software WAMS 1.0 with Module 1: Database, core system and availability and Module 2: Dredging Management have been developed in cooperation with Vienna University of Technologies - Institute of Transportation from 2012 to 2015 with the software being operational since July 2015. In the second stage viadonau teamed up with the principal developers (Hoffmann, M. & Haberl, A.) in order to extend the system with Module 3: Sediment Management, Module 4: Structure Management and Module 5: Traffic Management (Fig. 7) from 2016 to 2018.

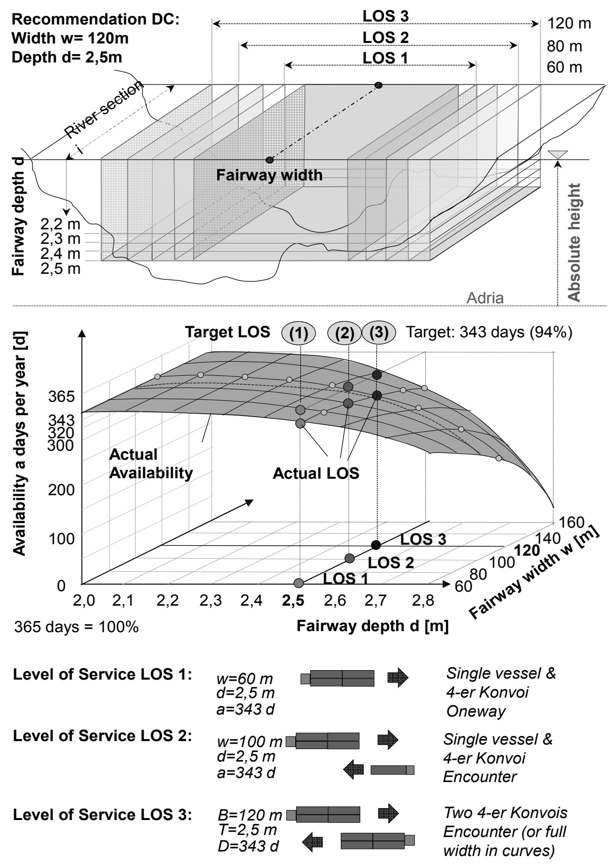


Figure 6: Fairway recommendations, total availability concept and levels of service (LOS) for a minimum fairway depth of 2.5 m

The focus of this paper is presenting an overview on principal considerations and methods together with developed functionalities and results from Module 5: Traffic Management in the WAMS software. The goal of Module 5 is to provide the waterway operator viadonau with all necessary information connecting the physical availability of the waterway as dynamic series system of fords, locks and civil structures with information from actual utilization of the vessel fleet in real time. In the following sections, the paper describes the anonymization of vessel trajectories as basis for generating comprehensive traffic analysis and fairway utilization information for any given set of conditions. Comparing cumulative vessel trajectories and traffic distribution for all types of encounters allows to optimize fairway alignment and pro-active maintenance measures for any given budget. Furthermore, these results will improve the capability of viadonau to provide shippers with actual and reliable information on fairway conditions as basis for optimizing transport logistics and competitive prices on the transport market.

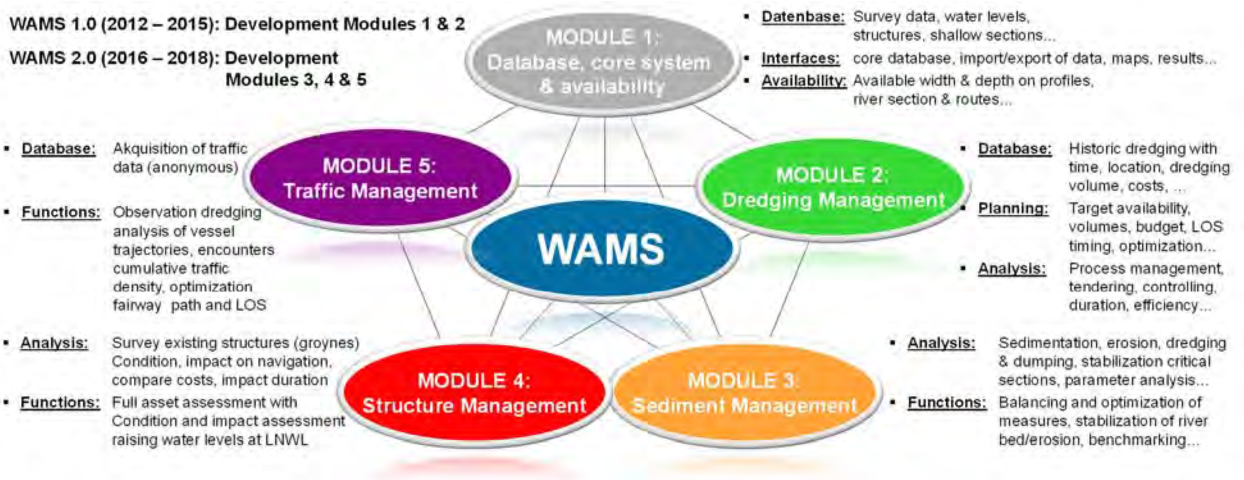


Figure 7: Overview developed modern modular Waterway Asset Management System WAMS of viadonau being operational since 2015

4. Traffic management and fairway alignment (Module 5)

4.1 Water depths, levels of service and fairway alignment

With Module 1: Database, core system and availability holding actual and historic information on riverbed surveys, water levels, structures, shallow sections and fairway alignments from almost a decade all kinds of analysis become feasible. As an example, Fig. 8 shows the calculation of total fairway availability as convex falling surface for increasing width and depth with the results for previously defined Levels of Service. From the perspective of economic waterway transport insufficient width translates into alternating one-way traffic at narrow sections with a few minutes waiting time in cases of encounters. In contrast, insufficient fairway depth at one single shallow section on an entire transport route will severely limit vessel utilization of the entire fleet with one-centimeter additional depth translating into a possible extra load of 7 to 20 tons depending on vessel or convoy type. With average travel times between one to three weeks on an average transport distance of 1.000 km neither waiting times at locks (on average 33 minutes) or rare

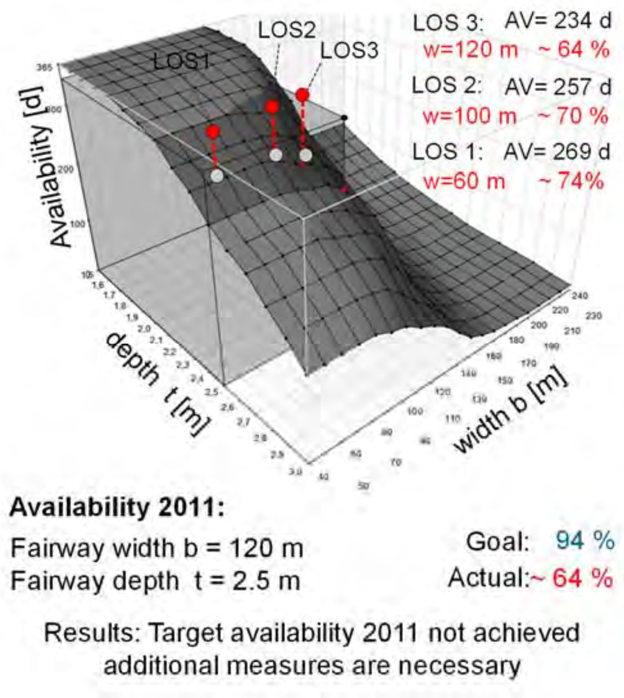


Figure 8: WAMS – Example calculation of total fairway availability of the River Danube with Levels of Service in 2011

cases of encounters in a few narrow sections really matter. However, conducting the same analysis on an inland waterway or a channel with high traffic density would yield different results and optimal strategies.

As a conclusion for the river Danube what really matters is providing a deep fairway channel with at least 60 m width and 2.5 m depth at all times on the entire transport route even at low water periods. The means of waterway authorities to achieve this goal are shifting of the fairway, dredging of shallow sections and the construction of structures (e.g. groynes, training walls). However, with a reactive approach to insufficient water levels possible realignments or emergency dredging would still lead to substantial losses of availability due to the time required for planning, tendering, implementation and communication of results. Employing pro-active strategies aiming at necessary measures prior to arriving at insufficient conditions lead to far better results. For implementing such pro-active strategies, the WAMS provides fast processing methods for an analysis of acquired data on all critical fords and water levels together with the capability for optimal fairway alignment (Fig. 9) and planning of dredging measures (Module 2). Conveying reliable availability information to skippers via signalization and different communication channels is equally important for success.

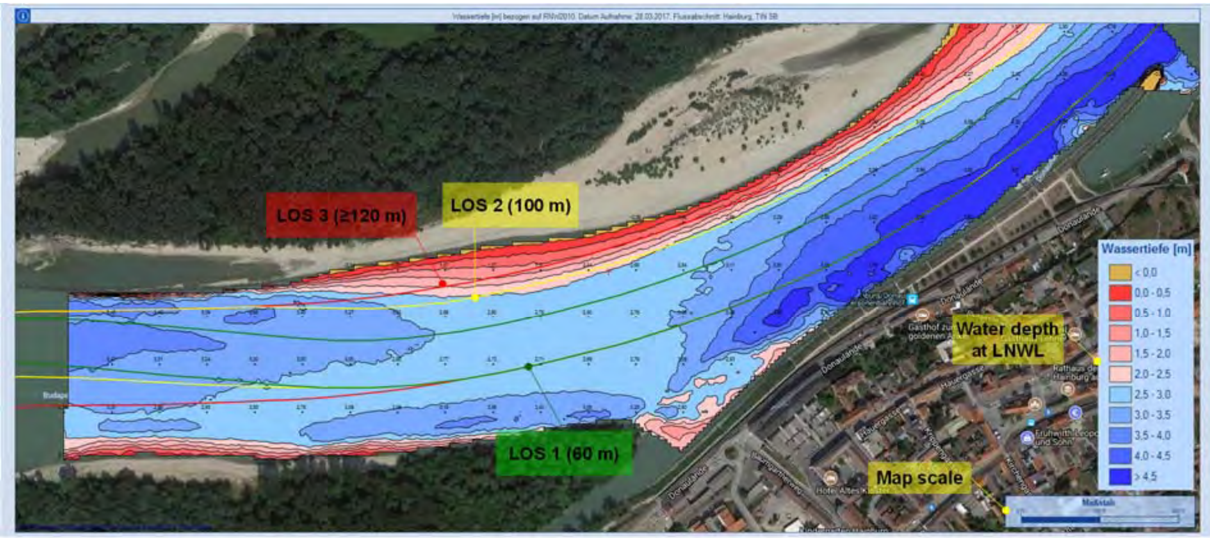


Figure 9: Automated analysis of water depths (LNWL 2010) based on multi-beam survey and water levels with fairway alignment and LOS 1, 2, 3 in 2017 (WAMS)

4.2 Vessel maneuvering, trajectories and encounters

In inland navigation, a transponder-based AIS (Automatic Identification System) is used for an automatic identification and tracking of vessels. This AIS transponder on vessels sends static (e.g. ship number, vessel type, number, name, call sign), dynamic (e.g. position, course, speed), and voyage related (e.g. draught loaded, destination) information to base stations on the shore. For reasons of privacy and data protection only the information on vessel type, position and draught loaded are kept in the WAMS for further analysis with all other data being discarded prior to importing. Furthermore, position data is thinned out reducing the size of the database and allowing for a fast processing.

Based on vessel types and recalculated heading information Module 5 of WAMS is capable of providing aggregated information on the number and types of vessels passing selected river sections in any given time-frame. Together with the distribution of vessel speed and loading depth this information enables a comparison of provided fairway availability and actual fairway utilization. For the analysis of vessel encounters it is necessary to project the position data and time stamp to a river kilometer with encounters due to passing or overtaking being located at the intersection of two-dimensional lines. Fig. 10 (top) provides an overview on all encounters for a selected river section and time frame with timing, location, direction, vessel speed and encounter type. By selecting a specific encounter, the trajectories of involved vessels are mapped, providing insight into maneuvers especially at low water periods (Fig. 10 bottom). At narrow sections, the aggregated information on encounters will allow an optimization of fairway alignment as well as an assessment of time losses and energy consumption due to waiting.

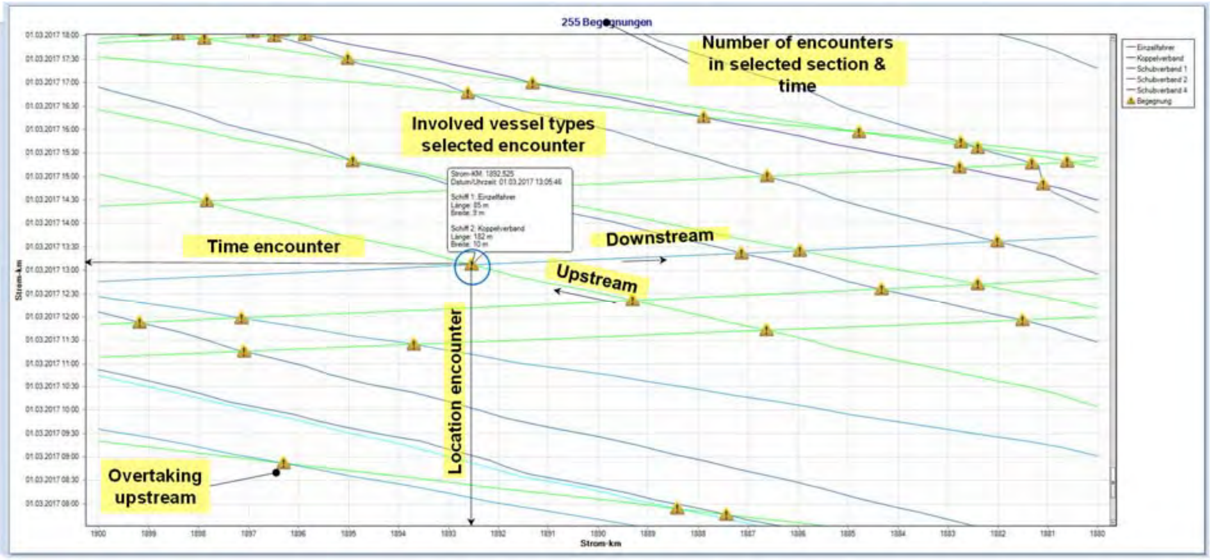


Figure 10: Automated traffic analysis and encounters in a river section (above) with vessel trajectories for a selected encounter (WAMS)

4.3 Traffic analysis, density and heatmaps

As interesting single vessel trajectories or encounters may be, they provide little information regarding optimal fairway alignment and optimization of fairway conditions for the entire vessel fleet. In this regard, the traffic distribution in comparison to fairway alignment, levels of service and actual water depths are what matters. For managing piers, ports and berths it may also be important to find out about their importance based on the number of arriving vessels and length of stay aside from their structural condition. These questions can be answered based on an algorithm using a defined grid with each square acting as a counting cell for vessel trajectories passing that square. For an analysis of piers, ports and berths this approach may be extended based on a predefined catchment area with an arrival counting if the length of stay from the transponder signal exceeds a certain threshold.

Fig. 11 (top) shows the results of this algorithm in the form of a color-coded traffic density heatmap in a given time frame and river section together with the calculated average fairway path and given level of confidence. The resulting heatmap may be used as an overlay for the map with water depths and fairway paths allowing for justified adaptions on a quantitative basis. Combining these results in cross sectional profiles also provides important insights as traffic distribution may be directly compared to water depths, fairway path and levels of service (Fig. 11 bottom). With skippers communicating accurate information on draught loaded it will also be possible to calculate underkeel clearance giving waterway authorities comprehensive information on actual utilization. A possible incentive for skippers conveying accurate data may be the possibility to use this info “a priori” in transport logistics and pricing and as feedback for loading efficiency on transport routes (Hoffmann M. et. al. 2016, 2018).

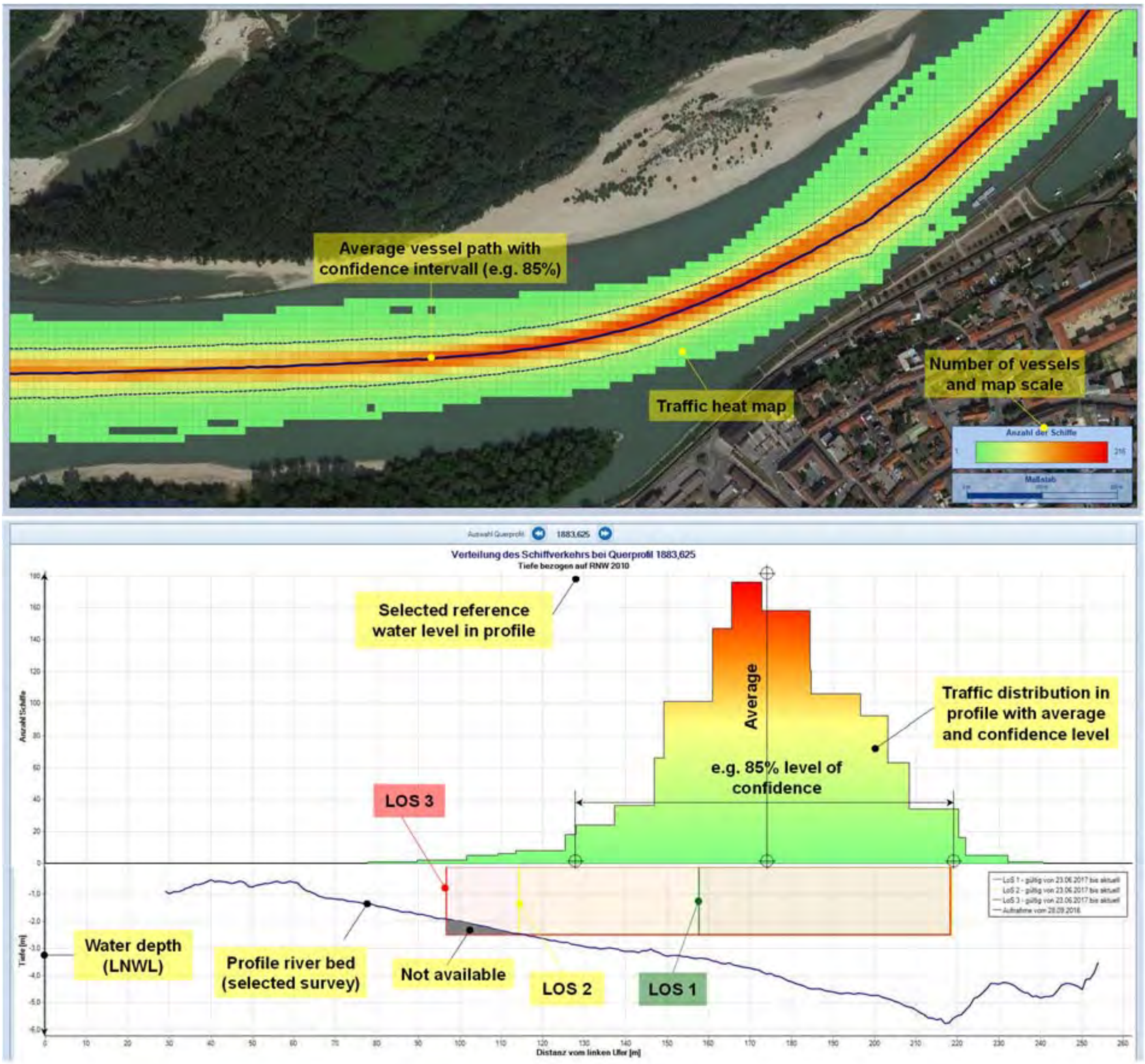


Figure 11: Automated traffic analysis in a river section (top) with heatmap and traffic distribution (bottom) in a selected river profile compared to water depths (WAMS)

4.4. Tracking of dredging vessels and achieved results

Monitoring the progress of fairway dredging as the most common maintenance measure on the upper Danube is of high importance for detecting any deviations during dredging as well as releasing updated plans and information on fairway availability. Starting with planning of dredging measures from Module 2: Dredging Management it is possible to track the entire dredging process including vessel trajectories between dredging and dumping sites. The transponder data in these cases is not part of the provided anonymous data for traffic analysis allowing specific tracking of the entire transport process including a playback of each transport run (Hoffmann M. et. al. 2016; 2018).

Fig. 12 shows detailed information on the selected dredging measure (e.g. river-km, time-frame, dredging volume, cost estimation, transport distance) together with an overview on vessel tracks on dredging and dumping sites as well as the entire route between. Riverbed surveys (multi-beam) shortly prior and after dredging ensure compliance with planned results being the basis for payments of dredging companies as well as releasing updated information on fairway availability to the shipping industry. The main advantage in this regard is streamlining the maintenance process towards a pro-active approach allowing fast reactions in cases of falling water levels and sedimentation in fords. Furthermore, acting on the base of pre-tendered framework contracts with dredging companies helps to avoid lengthy legal battles with unsuccessful bidders as well as negotiations under time pressure. With the WAMS being operational to some extent since July 2015 the first results of the implementation of these pro-active strategies are already visible leading to much higher efficiency of and accountability for investments together with an improved availability of the waterway Danube in Austria since 2016.

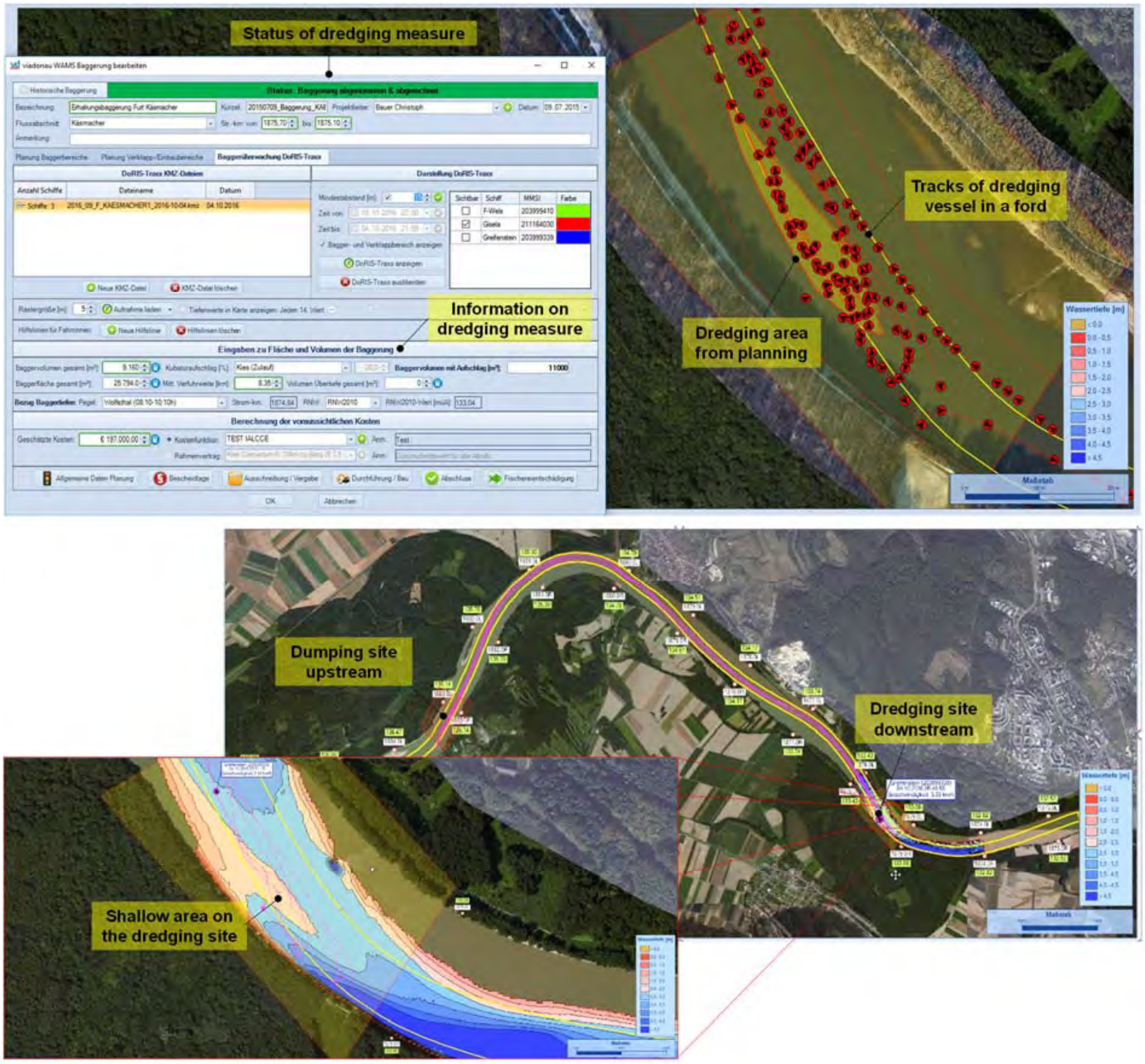
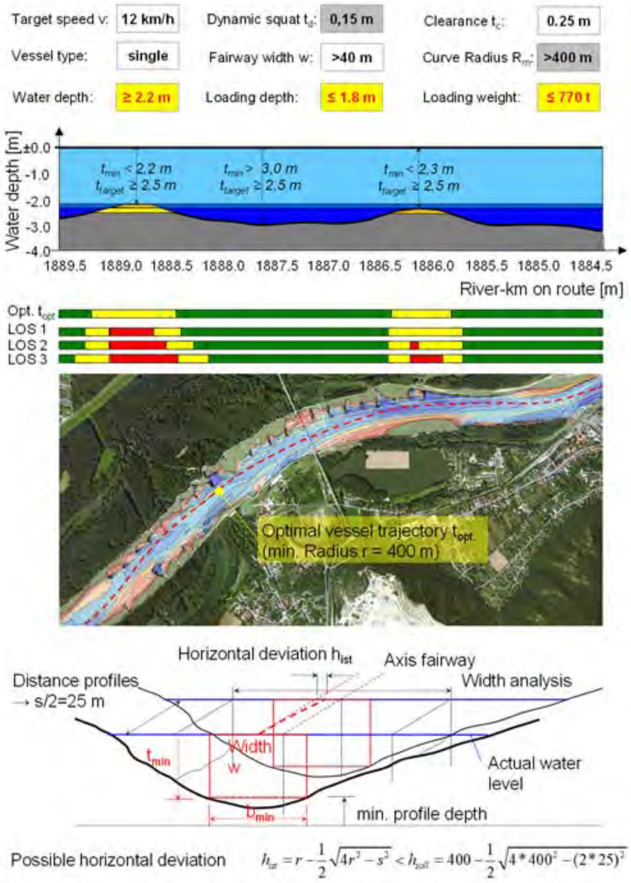


Figure 12: Automated analysis of dredging measure status with monitoring of dredging vessels and results based on transponder data and multi-beam surveys (WAMS)

4.5. Towards real-time availability and a waterway route planner

The availability of waterways is calculated as a serial system with the most critical ford regarding water depth at any time being the bottleneck for any given fairway width or level of service (LOS) on a transport route. The calculation of dive depths of vessels equals the sum of stationary draught loaded (speed v=0 km/h) and squat (v>0 km/h) with an additional clearance to avoid groundings for the necessary minimum water depth (viadonau 2013). The possible loading weight can be derived from calibration curves linking possible utilization and static draught loaded for different vessel types (Hoffmann M. et. al. 2014a,b). Combining this information, it is possible to estimate possible maximum vessel loading based on information of water depths or calculate an optimal trajectory from actual water levels, fixed points, minimum curve radii and tangent lines between cross-sectional profiles with specific search algorithms (Fig. 13). For the implementation the waterway route planner therefore has to incorporate current data from gauging stations and multi-beam surveys as well as the calibration curves for most common vessel types. Implementing data from a combination of echo-sounding and transponder information could add additional information regarding real-time assessment of current fairway availabilities.

Currently, it is already possible to calculate fairway availability based on existing fairway alignments or LOS, riverbed surveys and water levels for any given water depth on the entire Danube in Austria for the last 10 years until now. Fig. 14 shows such an automated WAMS - calculation for January 2016 with the critical fords at any given day resulting in a total availability for any selected condition. Together with the capabilities from Module 2: Dredging Management it is already possible to calculated dredging needs and costs on all critical sections for any given combination of fairway width and depth or available budget. The main challenge for extending already existing capabilities of the system towards a route and transport logistics planner for waterways would be reliable predictions of water levels and obtaining actual data from the entire Danube. In the past this was almost not possible as the survey standards have been very different. With the EU-project WAMOS the waterway authorities are currently working towards harmonizing information on water levels and riverbed survey in a common database. In the next years this database will be combined with actual depth information of echo sounders from the vessel fleet on the river Danube to provide the WAMS with route planning and transport logistics capabilities.



$$h_{ist} = r - \frac{1}{2}\sqrt{4r^2 - s^2} < h_{soll} = 400 - \frac{1}{2}\sqrt{4*400^2 - (2*25)^2}$$

Figure 13: Example for a simplified route planner with calculation of trajectory with max. water depth and possible loading of a vessel

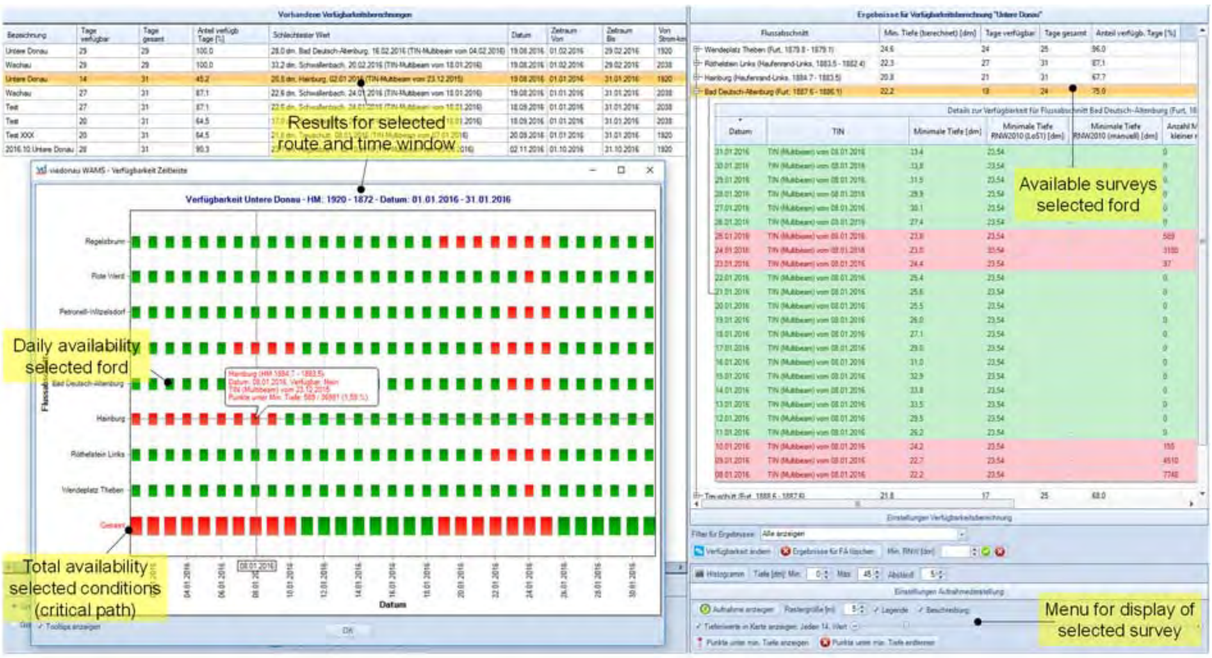


Figure 14: Automated analysis of available fairway width and depth with critical fords and total availability on a transport route (WAMS)

5. Conclusions and outlook

The analysis on goods transport in the Danube together with cost factors and infrastructure reliability clearly revealed why the majority of transport growth in the last two decades has been allocated to road transport with rail and waterway barely holding their ground. Enormous investments in road infrastructure, transport logistics and technology have led to substantial comparative improvements of road transport especially on short to medium transport distances. Existing bureaucratic hurdles, lack of investments, current low costs of fuel and external costs still not being internalized are also factors contributing to this development. Despite these developments waterway transport will stay competitive or may even regain market shares if continuous reliable fairway conditions with an available water depth of at least 2.5 m can be provided in low-water periods. Furthermore, improving transport logistics e.g. with fixed contracts using mixed modes of transport and optimal loading depending on actual conditions are necessary.

For managing a dynamic river as transport infrastructure in a cost-efficient and environmentally friendly way viadonau has teamed up with Vienna University of Technology and Hoffmann Consulting in order to develop a holistic Waterway Asset Management System (WAMS). In a first phase from 2012 to 2015 the principal methodological approaches for Modules 1: Database, core system and availability and Module 2: Dredging Management have been developed (WAMS 1.0). In the second phase from 2016 to 2018 Module 3: Sediment Management, Module 4: Structure Management and Module 5: Traffic Management are being implemented (WAMS 2.0). With the WAMS in operation since July 2015 the development is a work in progress providing constant feedback between theoretical considerations and practical results as new functionalities become available. In this regard, the WAMS is becoming an integrator of all kinds of data providing viadonau with the means to move from empiric reactive maintenance approaches towards quantitative asset management strategies with fast semi-automated processing capabilities and pro-active maintenance in a user-friendly environment.

Starting with an analysis of current developments in the transport market and the role of the waterway Danube this paper shows the principal approaches and functionalities of the WAMS with the main focus on methods and recently implemented functionalities in Module 5: Traffic management. The goal of this module is to provide viadonau with all necessary information connecting the physical availability and its optimization with an analysis of actual traffic flows and utilization of the vessel fleet in real time. To achieve this goal anonymized transponder data leaving only vessel type, position and draught loaded are imported for calculating encounters, traffic distributions and fairway utilization. Based on the implemented algorithms it is possible to generate traffic heatmaps and assess critical encounters in narrow sections at low water periods as a basis for aligning the fairway path and defined levels of service. Furthermore, the system is capable of monitoring the progress of pro-active dredging measures allowing a fast implementation and communication of results to the transport industry. With historic and actual data from riverbed surveys, water levels and traffic it is already possible to calculate the availability of any defined level of service for any transport route on the Danube in Austria in a matter of minutes. The possible loading of any vessel type can also be derived from calibration curves linking utilization and static draught with dynamic squat depending on vessel speed and necessary underkeel clearance.

The remaining challenges for optimizing vessel loading and route planning would be an implementation of prediction capabilities for water levels together with reliable information on fairway conditions on the entire Danube as average transport distances are rather long (1,000 km). With the Ministers of Transport on all riparian countries of the Danube endorsing a common Fairway Rehabilitation and Maintenance Master Plan in 2014 (FAIRway) the EU - project WAMOS will lead to one common database on fairway conditions of the entire Danube. Combining this information with actual water depths from echo sounders of the vessel fleet with already existing capabilities of the WAMS would improve both investment efficiency and enable modern transport logistics and route planning. For the years to come implementing these features in a user-friendly environment with automatic checking and updating will be one of the main upcoming challenges.

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River Information Services in a multimodal Intelligent Transport domain

by

Xavier Pascual[1].; Pedro S. Vila[2]; Cas Willems[3]

1 INTRODUCTION

Since the first initiatives of the European Commission on River Information Services, this concept on information exchange to support traffic and transport management in inland navigation, has found its way throughout the world. In recent years River Information Services (RIS) the development and especially the implementation of RIS has been considerable.

Multi- end Synchromodal transport and logistics will put new requirements on the RIS related services, systems, technology and standards. For RIS, this brings new opportunities for improving the quality and efficiency of Inland Waterway transport.

In the transport and logistics domain the focus is more and more on multimodal transport with information services in intermodal context. In this context important requirements are:

- A paper-free, electronic flow of information associating the physical flow of goods with a paperless trail built by ICT includes the ability to track and trace freight along its journey across transport modes
- The simplification of freight and transport information exchange to reduce the cost of transport.
- Freight should be identifiable and locatable regardless of the mode it is transported on.
- It is essential to create a single transport document for the carriage of goods in any mode.
- The increasing Reliability, Availability, Maintainability and Safety (RAMS) requirements of the systems supporting this services

It becomes more and more clear that digitization of transport and logistics is an essential prerequisite to guarantee in the coming decade an efficient and sustainable transport. Digitization has the objective to move from paper to electronic documents, through simplified procedures and integrated information exchanges across different sources.

A necessary condition is that standard (information) interfaces within the various transport modes are put in place and their interoperability across modes is assured.

RIS can contribute to above mentioned ambition and challenges if attention will be given to changing requirements on River Information Services in a multimodal environment and above all the focus will be on the seamless interfaces with information services in other transport modalities.

A key issue in this is to consider analyzing the interaction between RIS services with other concepts for the information services in other transport modes. In addition it can be of great benefit for the further development of RIS to consider using services, information, technologies, architecture, etc. that consists in the Intelligent Transport Systems (ITS) of these transport modes

A pro-active attitude towards this development of multi modal transport information services is essential. Development of multi modal information standards is demanded, Assessment of these future requirements should be put on the agenda of the RIS community. A transition strategy for RIS towards harmonized multi modal transport and logistics information services environment is to be developed.

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2 STATE-OF-THE-ART AND STANDARDS FOR RIS and ITS

As commented above, it has a wide spread of codes, standards and state-of-the-art definitions with regards to ITS, depending on its country or region of origin, its application (roads, railways, etc.), and others.

Although it is out of the scope of these paper to make an exhaustive compilation of all of existing reference standards, it is worth to note the ones with a greater range of applications or the corresponding to areas with a greater degree of development, and that has been the basis for the present compilation and compared analysis.

2.1 RIS standards

Without being exhaustive, an overview of key international rules and regulations that should be followed in case of the implementation and operation of River Information Services, is shown below:

i. Guidelines and recommendations for River Information Services (PIANC, WG 125, 2011)
ii. European Commission, Directive 2006/87/EC Technical requirements for inland waterway vessels, 2006
iii. Harmonised Commodity Description and Coding System of the WCO (world wide)
iv. UN Code for Trade and Transport Locations UN/LOCODE (world wide)
v. EDIFACT Standard of the UN (world wide)

2.2 Roads' ITS standards

Possibly the frame for Roads' ITSs is the most developed one, and where more innovations and efforts one could find involved. This fact, amongst others, is reflected in the existence not only of an extended bibliography, but also in the development of several national (and even regional like EU's one) standards that have finally led to the ISO 14813 (2015) – Intelligent Transport Systems, development.

Most relevant standards related to Roads ITSs considered for this Guidelines purposes are:

i. *Intelligent Transport Systems. Results from the transport research programme*. European Communities, 2001
ii. Directive 2010/40/EU of the European Parliament and of the Council of 7 July 2010 on the framework for the deployment of Intelligent Transport Systems in the field of road transport and for interfaces with other modes of transport
iii. U.S. National ITS Architecture. Intermodal Surface Transportation Efficiency Act of 1991
iv. European ITS Framework Architecture (informally called FRAME)
v. ITS Japan. Japanese ITS Architecture. (http://www.its-jp.org/)
vi. ISO 14813-1 Intelligent transport systems – Reference model architecture(s) for the ITS sector – Part 1: ITS service domains, service groups and services
vii. ISO 14813-3 Transport Information and Control Systems- System Architecture- Example Elaboration

As commented, yet from its first edition in 2007, the ISO 14813 has standardized the framework for Intelligent Transport Systems (ITS) in service domains and groups reflecting the evolution of technology-oriented transportation practices and applications. According ISO's defined framework, ITS are expected to address services in the following areas of the road transport domain:

In road traffic management	Traveler information	Electronic payment systems	Transport network operations and maintenance activities;	Freight mobility and inter-modal connectivity;
Multi-modal travel including both pre-trip and on-trip information and journey planning where the trip starts and/or finishes in the road transport domain;	Variable road pricing strategies for freight and personal travel;	Emergency and natural disaster-related response activities and coordination;	National security needs related to transportation infrastructure;	Cooperative-ITS.

Table 1. Service Domains for ITS according ISO 14813

ISO 14813 defines a framework based on ITS service domains, groups and services that serves as basis for developing ITS architectures and ITS-related concepts of operation, which in turn lead to the definition of the appropriate requirements, functionality and standards necessary to deploy specific ITS services.

Although ISO 14813 has been developed for road transport domain, as far as ITS are beginning to be applied in rail and maritime (RIS is an example) transport domains, it could be a proper reference standard at the time of developing the ones for other transport modes (as the RIS Guidelines are).

2.3 Railways

Unlike roads' domain, railways ITS developments haven't been accompanied by the conformation of a common standard that could provide some future applicable framework.

In the literature, one just could find some initial trials and common bodies of knowledge, or, like the EU case, a partial standardization effort on the frame of the European Rail Traffic Management System (ERMTS).

Considering the stated above, for these Guidelines purposes, the reference documents listed below have been considered:

i. *Intelligent Transport Systems for rail – A summary*. ITS United Kingdom, 2012 (www.its-uk.org.uk)
ii. ERTMS - European Rail Traffic Management System. © European Union, 1995-2017 (https://ec.europa.eu/transport/modes/rail/ertms_en)

Additionally to the previous documents, the experience obtained from the design and installation of several ITS systems and facilities throughout Europe and America has been considered for setting the definition and scope of the ITS services for railways.

Generally speaking, and again unlike road transport, a common body of definitions and standards hasn't been defined, resulting in a wide spread of terms and classifications for the services, systems and subsystems involved in ITS, depending on the concerned country or, even, the operating authority, who stablishes its own standards based on local experiences and operating requirements.

Additionally, and related to the previous, neither the concept of ITS services is clearly considered (unlike the ISO and/or RIS Guidelines do) for Railways, being most common the reference to systems or functionalities.

3 COMPARING RIS AND ITS

The PIANC working group on River Information Services did a restricted study on the ITS concept for roads and for railways (The ITS concept for railways is related, at some extent, to the Europe ERMTS - European Railway Traffic Management System). The aim of this study and consequently the presentation during the PIANC World Congress is to offer, firstly, an overview of the main features of the principal key objectives of ITS in comparison with RIS. Secondly a comparison focused on the services as provided by RIS and those of ITS for roads and ERTMS for railways. The presentation will give detailed information on the comparison between ITS road-services as defined in US, EU, Japan and ISO standards as well as a comparison of ERTMS Services for railways according several standards.

Based on the study, recommendations will be given regarding future interfaces with ITS development, or the extension of RIS services to other areas.

In order to obtain a reliable comparison according this standard purpose the features of ITS for roads and railways have been classified according the criteria considered for RIS and adapting, as much as possible, the concepts and definitions of roads and railways' ITS to them. Taking into account this classification criterion, a comparison considering the three levels of RIS definition: [1] Functional, [2] Services and [3] Technologies, will be made.

Finally, a classification will be given based on the Functions and Services established for RIS, and matching with them those concepts (both systems, functions, etc.) that best fit with their definitions.

3.1 Key Features comparison

WATERWAYS	ROADS	RAILWAYS
Oriented to Vehicle safety	Oriented to Safety	Highly oriented to the passenger => Active search for Service Improvement
Low interconnection with other modes	Medium level of intermodality	Interconnection / Interoperation / Intermodality
Fuel consumption optimization	Noise Reduction Fuel consumption reduction Alternative power systems (electrification)	Reduction of resources used to demonstrate environmental commitment
Public operated infrastructures Private operated fleets	Almost full Liberalization of infrastructures and fleets	Increasing competition between operators and gradual incorporation of new players (private or concessions)
N.D.	N.D.	Professionalization of the sector: Sale of associated services
Infrastructure Public management Private fleet management	Private management	Private management in public entities

Table 2. Key features comparison between RIS and ITS

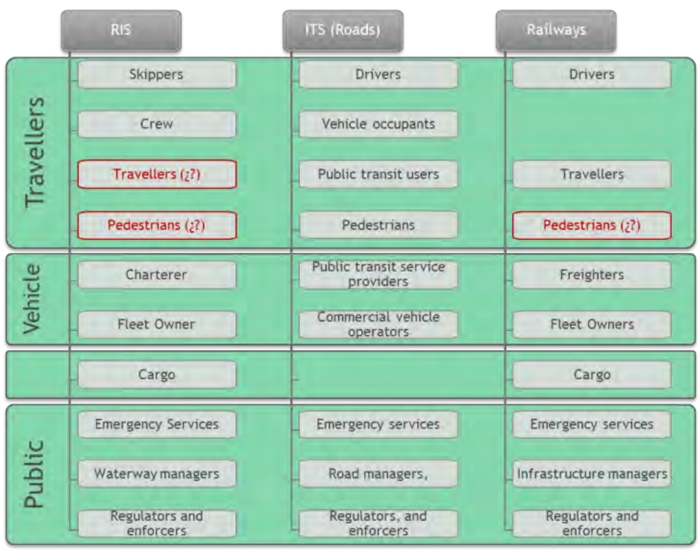

Table 3. Comparison of key stakeholders between RIS and ITS

3.2 Functional comparison

Although, from a general point of view, all Intelligent Transport Systems (considering RIS included in this category), should have, at the end, same or quite similar high-level functional requirements, it could be easily stated that actual features of the transport mode could made the owners to prioritize some of them above the rest.

This fact could be stated when comparing the Key functional requirements for RIS with the ones required to the Roads and Railways' ITS.

RIS	ITS ROADS	RAILWAYS
Improve inland navigation safety	Improve Operational safety	Improve Operational safety
Provide Traffic Information for safety and logistics	Provision of traffic data and statistics	**Sale of associated services**

RIS	ITS ROADS	RAILWAYS
Exchange information between vessels, locks, bridges, terminals and ports	Information exchange among infrastructure and assets: V2I (not only traffic but also transportation such as rolling stock interface towards PSD); V2V; V2G	Information exchange among infrastructure and assets:
Increase efficiency of use	Increasing Capacity through infrastructure **performance improvement** • operational efficiency (automatization processes) • economic efficiency (operation personnel optimization)	Increasing Capacity through infrastructure performance improvement **and management**
Environmental Protection	Energy Efficiency	Reduction of resources used to demonstrate environmental commitment
Integration of existing legacy systems	**Integration of existing legacy systems and operational procedures** **Integration of multisource and non-homogeneous technologies**	**Integration of existing legacy systems and operational procedures** **Integration of multisource and non-homogeneous technologies**
N.D.	**Dynamic adaptation to demand**	
N.D.	N.D.	**Active search for Service Improvement**
N.D.	N.D.	**Interconnection / Interoperation / Intermodality**

Table 4. Comparison of Key functional requirements for RIS and ITS

When comparing these systems, one could state that although most of indicated key functionalities are required to both of them, there are some differences in regards to the focus that is put in some areas.

It should be noted that this comparison is not exhaustive, and that has been made comparing non-homogeneous classifications. Hence, the fact that some functionality was not explicitly described for some environment (i.e. RIS or ITS), doesn't means that it hasn't been considered, but or it has been included, implicitly, in other concepts, or that doesn't have as much relevance than the others.

So, the shown comparison should be read in the sense that waterways RIS has a greater focus on environmental protection or data sharing between vehicles than ITS and, on the other hand, energy efficiency or demand adaptation has a greater impact in the case of roads and railways.

3.3 Services Comparison

As stated previously, in regards to ITS for roads there yet exists a wide body of knowledge and standards, developed from years ago, which has converged to a quite common framework that is, like the RIS case, based on the concept of Services.

The full services comparison analysis will be included in the incoming update of RIS guidelines. Although its extension exceeds the scope of the presente paper, in order to provide a general approach to performed comparison between these standards, a sample comparison is provided in the **¡Error! No se encuentra el origen de la referencia.** below, which compares ITS services as defined in US, EU, Japan and ISO standards for some of the services.

ISO 14813 (Roads) **ISO ITS Architecture Service Domains and Service Groupings** **Service domains**	**Rail ITS Services Envelope** **Rail ITS Services** **Services**	**RIS** **PIANC RIS Guidelines** **RIS Services and definitions**
N.D.	**Information to vehicle managers**	**Traffic Information (TI)**
N.D.	✓ Tactical Information ✓ Traffic Command & Control ✓ Rolling Stock Command & Control ✓ Strategic Information ✓ Dynamic traffic simulations (considering actual traffic and corridor conditions) ✓ Hourly program and calendar ✓ Machine - learning	Provision of information to network users 1. Tactical Traffic Information (TTI) 2. Strategic Traffic Information (STI)

ISO 14813 (Roads) **ISO ITS Architecture Service Domains and Service Groupings** **Service domains**	**Rail ITS Services Envelope** **Rail ITS Services** **Services**	**RIS** **PIANC RIS Guidelines** **RIS Services and definitions**
Traffic management and operations	**Traffic management system**	**Traffic Management (TM)**
Traffic control Transport-related incident management Demand management	European rail traffic management system (ERTMS) Automatic Train Location Train control Train communication Creation of trans-European traffic management facilities Stations management system Traffic Command & Control	Provision of management services Local Traffic Management (Vessel Traffic Services - VTS) Lock and Bridge Management (LBM) Traffic Planning (TP)
Traffic management and operations	**Maintenance & Construction (II)**	**N.D.**
Transport infrastructure maintenance management	Remote train maintenance management	

Table 5. Sample of services comparison between RIS and ITS for Roads and Railways

4 RAMS REQUIREMENTS

The increasing complexity of RIS infrastructure, the incoming increase of safety requirements (related with a quite more extensive use of waterborne transport and, the expected future use of unmanned vessels), the huge amount of input and output signals, the wide variety of failure functions of system's components, the complexity of functions relating SIL and failure probability, the restrictions on design based on the safe failure fraction, safety specification for different elements and the control of system failures as well as the own probabilistic nature of waterborne transport makes that RAMS requirements could be an incoming question to be afforded during planning and design stages of systems supporting RIS.

RAMS analysis (Reliability, Availability, Maintainability, and Safety), commonly used in other land transport ITS (Traffic Information Systems) infrastructures design (railways, highways, ports), could also be applied to inland waterway RIS (River Information Systems) environment, taking profit of its holistic scope for dimensioning operation and maintenance procedures taking into account not only functional and operational requirement, but also construction and design constraints.

The analyses are based on the use of the International Standard for Electrical, Electronic and Programmable Electronic Safety related systems (IEC 61508) as framework for the fully risk based approach for determining SIL (Safety Integrity Level) (SIL) requirements for those functions that are involved in the safety of the operations or in the achievement of the availability targets. This international standard uses a systems engineering approach the safety lifecycle as a framework in order to structure requirements relating to specification, design, integration, operation, maintenance, modification and decommissioning of the specific system.

5 DIFFERENCES AND SYNERGIES

Although RIS Guidelines are a well stablished and developed environment, and that from a general point of view, all Intelligent Transport Systems (considering RIS included in this category), should have, at the end, same or quite similar high-level functional requirements, it could be easily stated that actual features of the transport mode could made the stakeholders to prioritize some of them above the rest. This fact can be stated when comparing the Key functional requirements for RIS with the ones required to the Roads and Railways' ITS.

When comparing these systems, one could state that although most of indicated key functionalities are required to both of them, there are some differences in regards to the focus that is put in some areas pointed above. Therefore, and despite of the previous, being stated that still exists some room of improvement for RIS via cross feedback from ITS environment, main conclusion of developed work will be the proposal, included in WG 125 outcomes of a new working group oriented to develop a full comparison, focused in those developments in ITS domain that could best fit with inland navigation features, which would outline future RIS development trends.

Development of a ship eco-driving prototype for inland waterways

***F. Linde*[1]*, N. Huybrechts*[2], A. Ouahsine[3], *and P. Sergent*[4]**

ABSTRACT

The aim of this article is to develop a speed optimization software for inland navigation allowing to reduce fuel consumption by specifying a recommended sailing speed for each leg of the travel. For a given route, the water depth and currents are predicted with a 2D hydraulic model (Telemac 2D). Each leg of the route are then assigned a mean water depth and current velocity and resistance curves are obtained with a ship resistance model, based on a metamodel approximating CFD calculations or experimental results for ship resistance [Linde et al., 2015]. The fuel consumption is estimated with the model developed by Hidouche & Guitteny [2015]. The gradient projection algorithm [Rosen, 1960] is used to minimize the global fuel consumption for the itinerary. This model is used to simulate a real case: the itinerary of a 135 m self-propelled ship on river Seine, between Chatou and Poses (153 km). The optimized fuel consumption is compared with the non optimized fuel consumption, based on AIS speed data gathered on this itinerary. Different river discharges (low, medium and high) and sailing directions (upstream and downstream) are studied. The effects of the ship trajectory and travel duration on fuel consumption are also investigated.

1. INTRODUCTION

Inland waterway offers many advantages compared to roads and railways. The accident probability is very low and their cost in economic and human term is significantly reduced compared to the other means of goods transport. Inland waterways have little or no congestion and delay in goods delivery is hence reduced. A pushed barge with a load of 2 000 tons carries the equivalent of 50 railway cars at 40 tons each or 80 trucks at 25 tons each and therefore the carrying capacity per transport unit is very high. Moreover as pointed out by several studies [Rohacs and Simongati, 2007; Federal German Water and Shipping Administration, 2007; Agence de l'Environnement et de la Maitrise de l'Energie, 2006], inland waterway transport features as the most environmentally friendly mode. Making inland waterway transport more efficient and more sustainable is also one of the goals promoted by the European Commission through the NAIADES II package.

Even if the French waterway system is the longest in European Union with approximately 8800 km of navigable rivers and canals, French inland waterway transport (IWT) sector is lagging behind its European neighbors. IWT in France only represents 6% of all goods transport against 12% for Germany, 16% for Belgium and 33% for the Netherlands. One of the actions led by the French authorities to promote IWT is the construction of Seine-Nord Europe Canal. This new channel is expected to replace the existing Canal du Nord of limited capacity (barges of 250 to 650 tonnes) to form a major high capacity transport corridor for barges and push-tows up to 4400 tonnes, from Le Havre to Dunkirk, Benelux and the Rhine. Beside this hydraulic work, significant research efforts must still be conducted in order to improve inland vessel fuel efficiency. Indeed, inland navigation faces several challenges such as over-aging fleet, increasing fuel prices, climate change, and stronger environmental regulations regarding air emissions.

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The Lower Seine River is one of the main fluvial corridors in France. This river welcomes different ports at Le Havre, Rouen, and Gennevillier just downstream Paris. Each day, several vessels are travelling between Le Havre and Gennevillier. In order to reduce the fuel consumption and gas emission, it is proposed here to build a fluvial eco-driving prototype. Up to now, this prototype called "EcoNav" has been applied only on the Lower Seine River for one vessel type. The selected vessel type corresponds to a 135 m long and 11.4 m wide self-propelled vessel. This vessel type has been selected as their number is expected to increase in the next years and other projects are currently led on it as experimental tests or maneuverability studies. On board fuel monitoring is also planned in a near future for such vessel.

In the current paper, different solutions to reduce fuel consumption are first reminded and a literature review is proposed. The methodology inside the Econav model is then described. EcoNav model combines different sub-models as a 2D hydraulic model, a ship resistance model, a fuel consumption model and a nonlinear optimization algorithm to find optimal speed profile. Econav is used to simulate a real case: the itinerary of the self-propelled ship Oural (from Compagnie Fluviale de Transport) on river Seine, between Chatou and Poses (153 km). The optimized fuel consumption is compared with the non optimized fuel consumption, based on AIS speed profile retrieved on this itinerary. The effects of the ship trajectory and travel duration on fuel consumption are finally investigated.

2. Existing solutions for inland ship fuel consumption reduction

The average age of inland ships in Europe is above 40 years and a significant proportion of the current fleet is over-aged. Replacing the older ships with new units will take decades and therefore improving the economic and environmental performance of existing ships (retrofitting) is also necessary. The European FP7 project MoVe IT! [MoVe IT! FP7 European project, 2012] and the Danube Carpathian Programme [Radojfici´c, 2009] reviewed the existing solutions for improving the economic and environmental performance of existing or new inland vessels. These solutions can mainly be categorized into four main groups: (1) improvement in hull resistance; (2) improvement in propulsion and transmission efficiency; (3) improvement in propulsion plant; and (4) improvement of ship utilization (navigation).Those four categories are outlined in Figure 1 with solution examples.

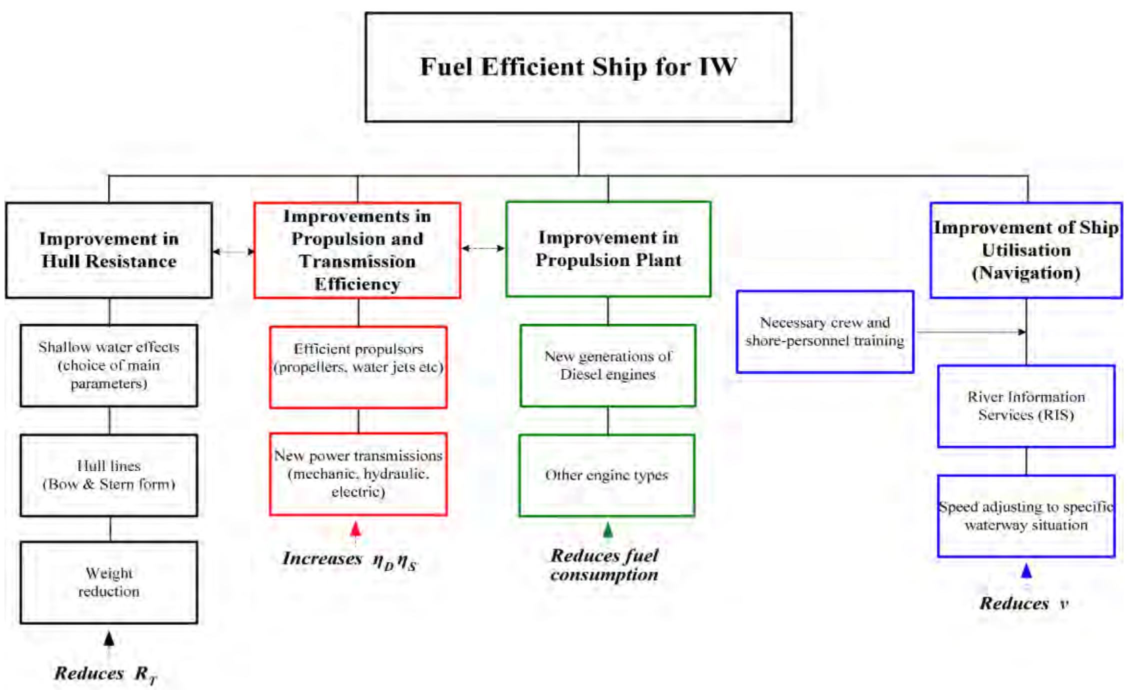


Figure 1: Main categories of retrofitting solutions [Radojčić, 2009]

2.1 Improvement in hull resistance

Decreasing hull resistance allows to reduce the forces opposing the movement of the ship and therefore leads to fuel consumption savings. Hull resistance can be decreased by adapting the hull shape to minimize resistance in shallow water. Hull shape optimization has mainly be focused on the design of efficient bow and stern regions [Rotteveel et al., 2014] which mostly contribute to wave and viscous pressure resistance. Zoelner [2003] showed that contemporary ships with improved designs have up to 50 % lower resistance than inland ships from a few decades ago. Another way to reduce hull resistance is to make the ship hull lighter. The materials used for inland ship construction is almost exclusively steel because of its durability. However, the use of lightweight materials and structural arrangements (such as reinforced composites materials or sandwich structure) for shipbuilding [Noury et al., 2002] could lead to significant fuel consumption reduction. For instance, Jastrzebski et al. [2003] reported a structural weight saving of 40 % if steel sandwich panels would be used for the construction of a small barge of 32.5 m. Solutions intending to decrease the frictional resistance of the hull have also been studied. For instance, the frictional drag can be reduced by using air as a lubricant with techniques such as injecting air bubbles in the boundary layer, using air films along the bottom plating or air cavities in the ship's bottom [Foeth, 2008]. It is also possible to reduce the ship resistance by applying special coatings with anti-fouling properties allowing to reduce water friction [Stenzel et al., 2011].

2.2 Improvement in propulsion and transmission efficiency

Many existing inland ships are built with conventional propellers whose efficiency often reach 70 % in maritime navigation but can be as low as 20-40 % when used in restricted waterways [Georgakaki and Sorenson, 2004]. Replacing those conventional propellers with one more suited to inland navigation could lead to significant fuel consumption reduction. For example, Geerts et al. [2010] reported that the three blade propeller of the Campine-Barge 'Prima' was replaced with a five-blade propeller which led to a speed gain of 1 km/h for the same fuel consumption. Many examples of innovative propellers more suited to inland navigation exist [MoVe IT! FP7 European project, 2012]: ducted propeller with a non-rotating nozzle which deliver greater thrust; adjustable tunnel preventing the income of air at low draft; pre swirl stator redirecting the flow before it enters the propeller disc; skew or contra-rotating propellers.

2.3 Improvement in propulsion plant

Currently, diesel engines are the most common types of engines used for inland ships. However, those engines are often marinized general application diesel engines or truck engines. Those engines are also usually much older than those used for road transport and belong to previous technological generation (inland ship engine have a lifetime of 20 years against 5 years for truck engines). Therefore, inland ships emit non-negligible quantities of carbon dioxide (CO_2), sulphur-oxides (Sox), nitrogen-oxides (NOx) and particulate matter (PM). With increased environmental legislation on transport emissions, inland shipping will need to reduce its greenhouse gas emissions and can benefit from the use of alternative fuels or im proved diesel engines. Possible solutions for improvement in propulsions plants include [MoVe IT! FP7 European project, 2012]:

• diesel electric propulsion where diesel powered generators provide electrical power used to propel the ship;

• hybrid propulsion using more than one power source to propel the ship (diesel generator with batteries for instance);

• natural gas engines using liquefied natural gas (LNG) instead of ordinary fuel;

• multi (truck) diesel electric using several truck diesel engines as generators in a diesel electric propulsion;

• fuel cell converting the chemical energy of a fuel (hydrogen or natural gas for instance) into direct current power.

2.4 Improvement of ship utilization (navigation)

Improvement in ship operations, aiming to reduce and/or adjust the ship speed during a travel can also lead to fuel consumption reduction. For instance, a traffic control system indicating the availability of locks and quays to ship operators could help them adjust their speed during the travel in order to minimize fuel consumption while ensuring to respect their ETA. River information services (RIS) offering possibilities for voyage planning, tracking and tracing through rapid electronic data transfer (in real-time) can contribute to a safe and efficient transport process and lead to a reduction of fuel consumption. Replacing the smaller fleet units or in creasing the ship main dimensions can help to lower the emissions per tonnekm. Prediction of the water depth on the ship route can help to adjust the ship speed in the shallow water zones. Applying slow steaming, which consist in sailing at a reduced speed, can also lead to consumption reduction.

2.5 Slow steaming and speed optimization

A ship sailing at a reduced speed will emit less greenhouse gas and consume less fuel. This practice, also known as slow steaming, is already used in many maritime commercial ship sectors such as tankers, bulk carriers and containerships, but rarely applied for inland navigation. A basic application of slow steaming consists in sailing at a speed lower than the vessel's design speed. More evolved slow steaming practices involve speed optimization algorithm taking account of several factors (weather forecast, current, trim, draft and water depth) [Psaraftis and Kontovas, 2014]. Some industrial products such as Eniram speed[1] already exist and are frequently used for maritime navigation. However, to the knowledge of the author, no such products exist or are used for inland navigation. A prototype version of the EconomyPlanner is currently developed and tested within the framework of the FP7 Eu project MoVeIT! (Bons et al., 2014). The aim of the EconomyPlanner is to generate a real time local water depth map through cooperative depth measurements and determine the optimal track and vessel speed respecting ETA (expected time of arrival) conditions for a given itinerary in order to reduce fuel consumption. The optimization of the fuel consumption is carried out by a module named Virtual Ship and developed by MARIN (Maritime Research Institute Netherlands). The power and resistance calculations are based on formulas derived from regression analysis on model experiments carried out at MARIN and sea trials. Corrections of shallow water effect are made based on Schlichting (1934) and Landweber (1939) methods.

3. Model development

Through an optimization algorithm minimizing the fuel consumption, the EcoNav model looks for the best speed profile for a given itinerary with operating conditions and under specified constraints. The constraint used in the optimization process is the maximum travel duration T_{max}. The model for fuel consumption FC_T computes the fuel specific consumption corresponding to the thrust input necessary to counteract the ship resistance. This ship resistance represents the hydrodynamic force R_T opposing the ship movement for given conditions (ship's load, channel geometry conditions, ship's speed, and current velocity). Finally, the operating conditions are defined by the parameters describing the hydrodynamics conditions in which the ship will sail on the itinerary. These conditions are the channel width W, the water depth H and the current velocity U . The last two quantities are predicted by using a 2D hydrodynamic model (Telemac2D V7P0). Figure 2 illustrates the working principle of EcoNav.

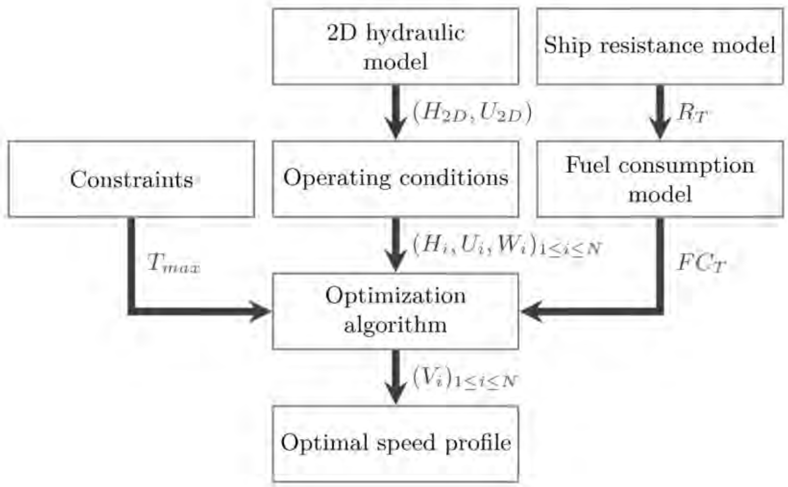


Figure 2: EcoNav flowchart

Each module is described in the following sections.

3.1 Ship resistance model

The fuel consumption of the ship is directly linked to the effective towing power $P_E = R_T \times V$ required to move the ship at a constant speed V. The ship resistance R_T can be evaluated in different ways: empirical formulae, CFD (Computational Fluid Dynamics) model or experimental data. As pointed out by Linde et al [2017], empirical formulae are not efficient in restricted waterways especially for narrow section. It is thus proposed to rather work with a surrogate model built from sampled data. The surrogate model could be fed with CFD results, experimental data or a combination of both of them. Here for the studied vessel, several experimental data have been conducted at Anast Lab from Liege University Belgium [Linde et al 2017]. It is thus chosen to only feed the surrogate model with these experimental data.

In order to avoid repeated use of computationally expensive simulations or costly experiments, surrogate models are often used to provide rapid approximations of more expensive models. These models are used in the engineering community for a wide range of application (Koziel and Leifsson, 2013).

The surrogate modelling approach approximates the simulation or experimental data $f_p(\boldsymbol{x})$ with the surrogate model output $\widehat{f_p}(\boldsymbol{x})$:

$$f_p(\boldsymbol{x}) = \widehat{f_p}(\boldsymbol{x}) + \varepsilon\,(\boldsymbol{x}) \tag{1}$$

where $\boldsymbol{x}$ is the coordinate vector where the function is evaluated and $\varepsilon(\boldsymbol{x})$ is the approximation error.

The governing parameters used for this surrogate model are the water depth H to ships draught T ratio H/T (quantifying the water depth restriction); the channel width W to ships breadth B ratio W/B (characterizing the channel width restriction) and the vessels speed V. It is worth mentioning that those parameters are independent and characterize the three main factors who have an effect on ship resistance in restricted waterway. As a result, the ship total resistance RT is expressed as follows:

$$R_T = f\left(V, \frac{H}{T}, \frac{W}{B}\right) = f(\boldsymbol{X}) = \widehat{R_T} + \epsilon \tag{2}$$

where $\boldsymbol{X} = (V, \frac{H}{T}, \frac{W}{B})$ is the coordinate vector and $\widehat{R_T}$ is the approximation function of R_T given by the surrogate model.

Popular surrogate model techniques (Queipo et al. 2005, Forrester and Keane 2009; Simpson et al. 2008) include ordinary least square (LSM), moving least square (MLS), Kriging, support vector regression (SVR) and radial basis functions (RBF). Different approaches have been tested and their accuracies have been compared via the mean square error (MSE):

$$MSE = \frac{1}{n}\sum_{i=1}^{n}\left(R_{Ti} - \hat{R}_{Ti}\right)^2 \tag{3}$$

Where $\left(\hat{R}_{Ti} = \hat{R}_T(\boldsymbol{X_i})\right)_{i=1..n}$ are the n predictions of the n observed data points $\left(R_{Ti} = R_T(\boldsymbol{X_i})\right)_{i=1..n}$.

3.2 Ship fuel consumption model

To evaluate fuel consumption, it is necessary to calculate the break power P_B delivered by the main engine to move the ship at a speed V. However, this power is greater than the effective towing power $P_E = R_T \times V$ because of the various energy losses occurring in the ship propulsion. The main components of ship propulsion are:

- a prime mover engine transforming an energy source into rotational mechanical energy;
- a reduction gear reducing the high rotation speed of the engine;
- a main shaft supported and held in alignment by bearings and transmitting the rotational mechanical energy from the reduction gear to the propeller;
- a propeller converting rotational motion into thrust by imparting velocity to a column of water and moving it in the opposite direction of the ship movement.

The energy loss occurring between each energy transformation is quantified through efficiencies:

- the hull efficiency $\eta_H = P_E/P_T$ is the ratio between the effective power P_E and the thrust power P_T delivered by the propeller to the water,
- the propeller efficiency $\eta_B = P_T/P_B$ is the ratio between the thrust power P_T and the power delivered to the propeller P_B;
- the shaft efficiency $\eta_S = P_D/P_B$ is the ratio between the power P_D delivered to the propeller and the brake power P_B delivered by the main engine.

The global propulsion efficiency η_G is defined as the product of the three efficiencies described above:

$$\eta_G = \eta_H \times \eta_B \times \eta_S \tag{4}$$

Figure 3 illustrates the performance quantification of a typical ship propulsion.

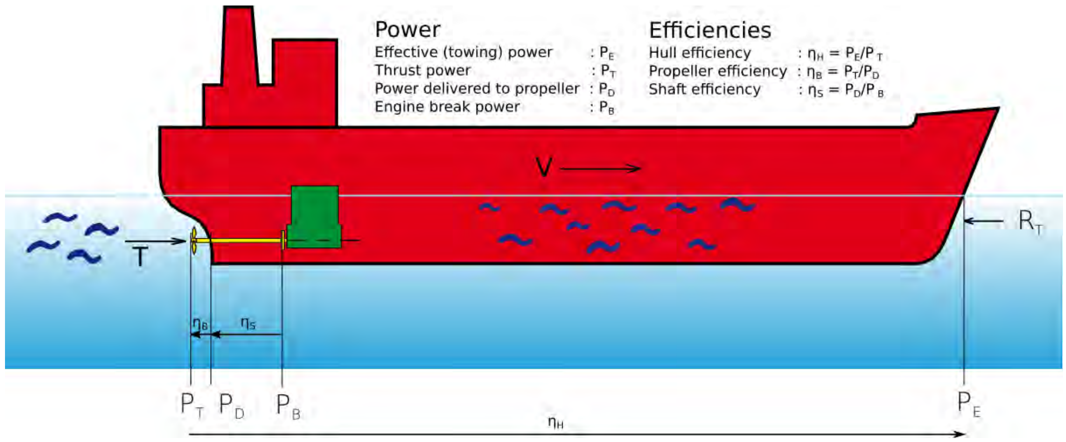


Figure 3: Performance of a typical ship propulsion [adapted from MAN, 2011]

The link between the breakpower P_B delivered by the main engine and the effective power P_E can be written as:

$$P_B = \frac{1}{\eta_H \times \eta_B \times \eta_S} P_E = \frac{1}{\eta_G} P_E \tag{5}$$

From the engine break power P_B [kW], the fuel consumption rate $\dot{m}_f$ [kg/h] is calculated through the specific fuel consumption SFC [kg/kW/h] (Eq.)

$$SFC = \frac{\dot{m}_f}{P_B} \tag{6}$$

The global propulsion efficiency η_G is taken equal to 0.5 which corresponds to an average value observed for inland vessels [Hidouche et al., 2015]. However, this estimation could be more accurate if each performance ratio is detailed, especially the propeller efficiency, but this implementation needs other parameters (propeller characteristics, hull shape,...) .

The specific fuel consumption model is based on a regression analysis of specific fuel consumption curves against power ratio (P_B/P_{max}). The specific fuel consumption data from recent and representative engine were collected from the manufacturers (mainly Cummins, MAN, Caterpillar and Wartsila). Furthermore, the declination of the model by engines power class and the split of the model in two zones of regression (power regression for zone 1 and polynomial regression for zone 2) provide a better accuracy to this model.

Figure 4 illustrates the regression analysis and Table 1 summarizes the equations of the ship consumption model and the error range.

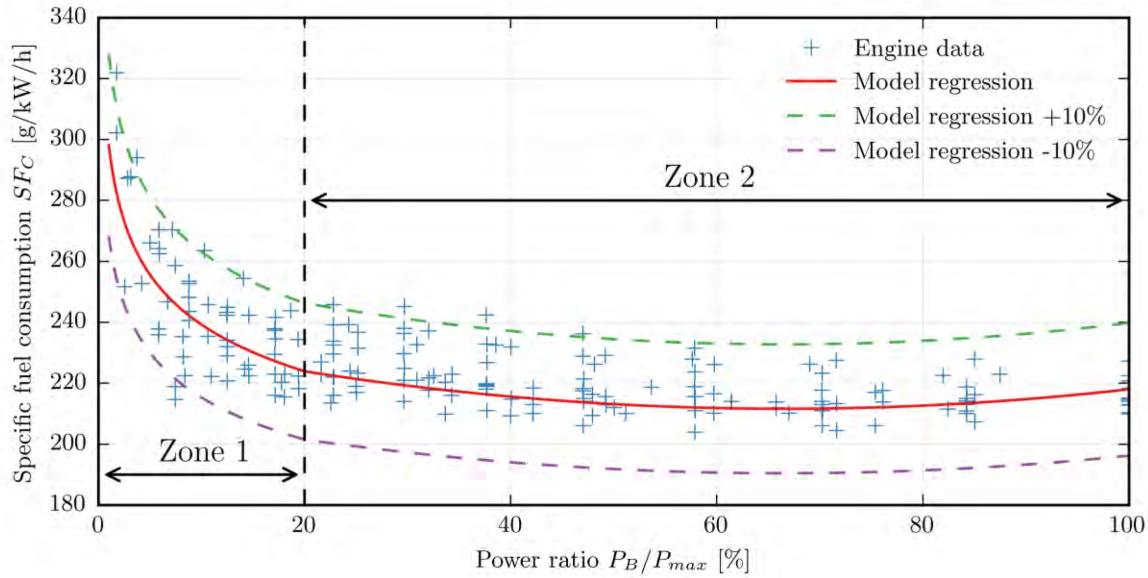


Figure 4: Specific fuel consumption model for 1000kW-2000kW range; model curve and engine data (adapted from Hidouche and Guitteny [2015])

$\mathbf{P_{max}}$ [kW]	$\mathbf{X = P_B/P_{max}}$ [%]	$\mathbf{SFC = f(X)}$ [g/kW/h]	**Error** [%]
100-300	$0-20$	$398.89(X)^{-0.1987}+8.945$	10
	$20-100$	$242.51-0.810(X)+0.0065(X)^2$	7
300-500	$0-20$	$342.077(X)^{-0.1361}$	10
	$20-100$	$237.84-0.5957(X)+0.0040(X)^2$	7
500-1000	$0-20$	$327.708(X)^{-0.1262}+1.984$	15
	$20-100$	$230.192-0.4496(X)+0.0033(X)^2$	10
1000-2000	$0-20$	$296.346(X)^{-0.0963}-1.06$	10
	$20-100$	$236.786-0.7577(X)+0.0064(X)^2$	10
2000-10000	$0-30$	$265.583(X)^{-0.0570}-1.743$	7
	$30-100$	$240.204-0.9639(X)+0.0064(X)^2$	5
> 10000	$0-30$	$218.92(X)^{-0.0570}-1.4368$	-
	$30-100$	$198-0.7945(X)+0.0053(X)^2$	5

Table 1: Specific fuel consumption model equations and errors

3.3 2D hydraulic model

Flow characteristics must be provided along the vessel journey. Up to now, only the fluvial part of the Lower Seine River from Gennevillier to Poses has been considered. The Seine estuary has not been treated yet. The river is split into 4 reaches delimited by weirs and locks: between Chatou and Andresy (reach 1); between Andresy and Mericourt (reach 2); between Mericourt and Notre Dame La Garenne (reach 3); between Notre Dame La Garenne and Poses (reach 4).

On each fluvial reach, hydraulic models have been built on Telemac 2d (www.opentelemac.org) which solves the Saint-Venant equations using the finite-element method on unstructured grid [Hervouet, 2007]. Mesh size comprises within 60 000 to 120 000 nodes according to the reach length and the numbers of isle inside the reach. Distance between the mesh nodes varies between 3 m (typically in ship locks) and 12.5 m (typically in the middle of the reach). For each reach, measured discharge is imposed at the upstream boundaries and measured water level is set for the downstream boundary. Friction coefficient is calibrated using measured water level at the upstream boundary.

The data used for this model is the December 2012 river freshet. The average discharge for Seine River between 2008 and 2015 is 480 m^3/s. In December 2012, the river freshet started with low discharge (< 200 m^3/s) during the first few days, then a first increase in discharge was observed (up to 500 m^3/s) and a final surge in discharge (up to 950 - 1000 m^3/s) occurred near the end of the event. Therefore, this event allows to simulate Seine river hydrodynamic conditions for three characteristic discharges (200, 500 and 900 200 m^3/s). From these 2D hydraulic models, values of the water depth and flow velocities are extracted along the vessel path for different flowrate values.

3.4 Operating conditions

The itinerary on which the ship speed is optimized is characterized by a set of parameters called operating condition. These parameters are the channel width W, the water depth H and the current velocity U and are required in order to calculate the ship resistance with the model described in section 3.1. From 2D hydraulic models, water depth H, current velocity U and river width W are extracted every 10 m on the vessel trajectory. The itinerary is then approximated by n legs of length $li = 10\,m$ and characterized by the parameters $(H_i;\ U_i; W_i)_{1\le i\le n}$. This itinerary is then further simplified for the optimization process by merging the n fine legs into N coarser legs LCi of length L_i, by using the Piecewise Aggregate Approximation technique (Keogh et al., 2001).

If $X = [x_1, \dots, x_n]_{1\le i\le n}$ is the list of parameters (x denotes either the water depth H, the current velocity U or the river width W) extracted every 10 m, the data $Y = [y_1, \dots, y_n]_{1\le i\le n}$ characterizing the N coarser legs is calculated as:

$$y_i = \frac{N}{n} \sum_{j=\frac{n}{N}(i-1)+1}^{\frac{n}{N}i} x_j \tag{7}$$

3.5 Optimization algorithm

The optimization algorithm minimizes the global fuel consumption for the itinerary by finding the optimal speed at which the ship should sail on each leg. The quantity of fuel FCi (kg) consumed by the ship on leg LCi of length Li (km) is given by:

$$FC_i = \dot{m}_{fi} \times \Delta T_i = SFC_i \times P_{Bi} \times \Delta T_i \tag{8}$$

where ΔT_i (h) is the time necessary for the ship to cover the distance L_i.

It is assumed that ship sails at constant speed V_i on leg LCi, therefore $\Delta T_i = L_i/V_i$. Equation (9) also gives $P_B = \frac{1}{\eta_H \times \eta_B \times \eta_S} P_E = \frac{R_{Ti} \times V_i}{\eta_H \times \eta_B \times \eta_S}$. As a result the quantity of fuel FC_i can be written:

$$FC_i = \frac{SFC_i \times R_{Ti} \times L_i}{\eta_H \times \eta_B \times \eta_S} \tag{10}$$

The total fuel consumption FC_T on the itinerary is then given by:

$$FC_T = \sum_{i=0}^{N} FC_i \tag{11}$$

The expression FC_T thus defined is a non-linear continuous function of variable $\boldsymbol{V} = (V_0, \dots, V_N)$.

The formulation of the optimization problem can then be written as:

$$\begin{aligned} &\textbf{minimize} && FC_T(\mathbf{V}) \\ &\textbf{such that} && \sum_{i=0}^{N_C} \frac{L_i}{V_i} \leq T_{max} \\ & && V_i > 0 \qquad\qquad i = 0, \dots, \mathrm{n} \end{aligned} \tag{12}$$

The first constraint set a maximum travel duration and the other constraints are only set to restrict the speed values to positive values. This optimization problem is a non-linear optimization problem with nonlinear constraints. Several methods are available to solve this type of optimization problem such as penalty function method, gradient projection method, feasible directions method and multiplier methods. However, these methods often perform better for linear constraints. For this reason, the optimization problem is reformulated as follows:

$$\begin{aligned} &\textbf{minimize} && FC_T^*(\mathbf{X}) \\ &\textbf{such that} && \sum_{i=0}^{N_C} L_i \times X_i \leq T_{max} \\ & && \mathrm{X_i} > 0 \qquad\qquad i = 0, \dots, \mathrm{n} \end{aligned} \tag{13}$$

where $\boldsymbol{X} = \frac{1}{\boldsymbol{V}} = \left(\frac{1}{V_0}, \dots, \frac{1}{V_N}\right)$ and $FC_T{}^*(\boldsymbol{X}) = FC_T\left(\frac{1}{\boldsymbol{X}}\right) = FC_T(\boldsymbol{V})$. With this formulation, the problem is now a non-linear optimization problem with linear constraints in $\boldsymbol{X}$.

4. Application to a 135 m long vessel sailing on the Lower Seine River

In the following sections, more details are provided on the results of the surrogate model for the ship resistance and on the optimization techniques. Then the speed optimization is applied to a 135 m long vessel on the Lower Seine River.

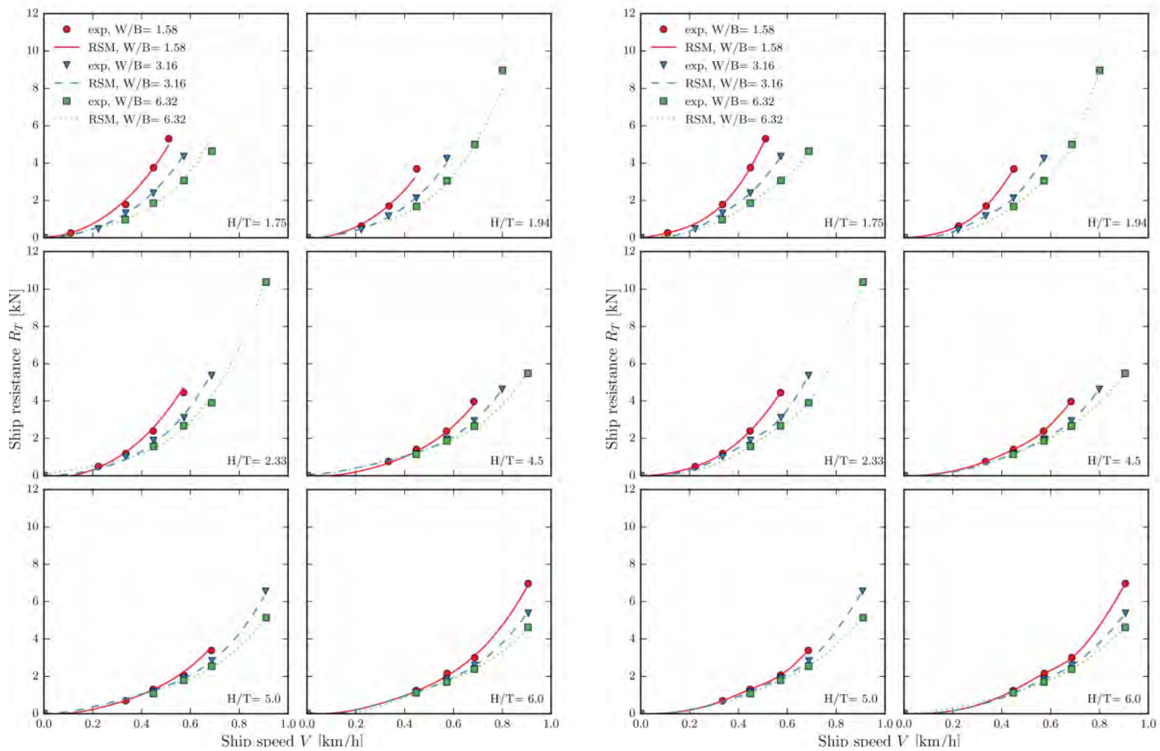
4.1 Surrogate model for ship resistance

Five different approaches have been tested: polynomial regression (PR), moving least square (MLS), Kriging, support vector regression (SVR) and radial basis functions (RBF). For these five methods, the surrogate model results have been compared to the Anast experimental results and the RMS has been calculated for each method. The RMS results are presented in Table 2:

Method	RMS
PR	0.2
MLS	0.04
Kriging	0.03
SVR	0.04
RBF	0

Table 2 RMS values obtained for the 5 tested methods

The mean square error calculated for the four tested methods showed that overall; MLS, Kriging, SVR and RBF methods give more accurate predictions than the PR method. It should also be mentioned that the RMS value for the RBF method is equal to 0 because this method in an interpolating method. Figure 5 illustrates the comparison between the experimental data (markers) and the surrogate models (lines) for the Kriging and RBF methods.

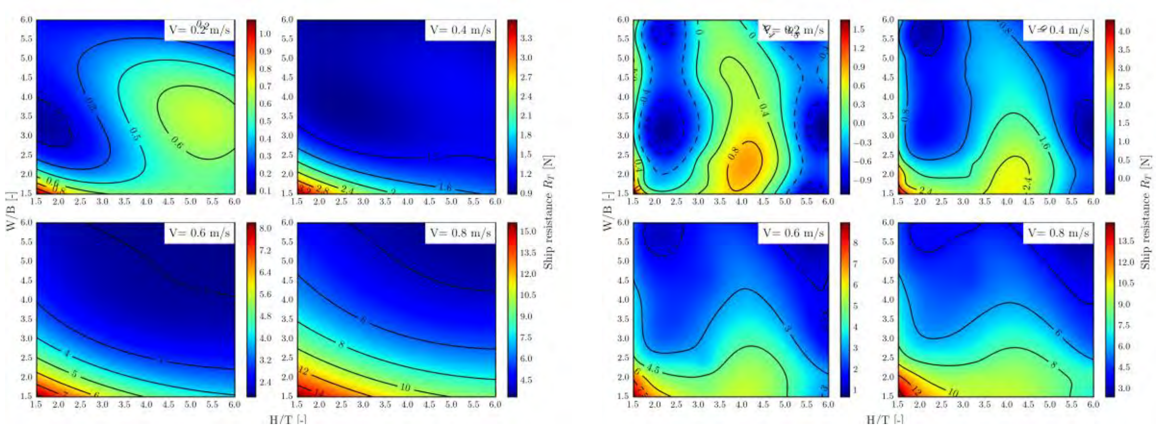


(a) (b)

Figure 5: Comparison between experimental data and surrogate model output for (a) Kriging and (b) RBF methods

Figure 5 shows that both methods give accurate predictions for ship resistance. However, the evolution of the ship resistance curves is smoother in the case of the Kriging method than in the case of the RBF method.

Figure 6 shows the iso-contours of ship resistance in function of W/B and H/T at four different ship speeds (V=0.2, 0.4, 0.6 and 0.8 m/s) calculated with the Kriging and RBF method.



(a) (b)

Figure 6 Iso-contours of ship resistance in function of W/B and H/T ratios at V=0.2,0.4, 0.6 and 0.8 m/s calculated with the (a) Kriging and (b) RBF methods

Figure 6 shows that the iso-contours of ship resistance obtained with RBF method are fairly irregular and this behavior does not represent a physical evolution of the ship resistance. The same observations were made for the SVR method as well. However, the iso-contours of ship resistance calculated with the MLS and Kriging method showed a regular evolution, as illustrated in Figure 6 (a). Therefore, the MLS and Kriging methods are more adapted for this surrogate model. However, the MLS method is computationally more expensive than the Kriging method because the approximation coefficients have to be calculated for each prediction. The optimization process needing many function evaluations, the Kriging method is chosen for this surrogate model as it is computationally quicker than the MLS method.

4.2 Comparison of the optimization techniques

The four optimization techniques have been tested with a ship sailing upstream on Reach 1 for a discharge of 200 m³/s. 50 random uniformly distributed samples for initial speed distribution have been generated in the bounded region defined by the optimization problem (see Equation 12). This random sampling is based on the Billiard Walk algorithm [Gryazina and Polyak, 2014]. Each optimization technique has been run on this random sample and the average converging time to the solution is calculated. Table 3 shows the average number of iteration $Niter$, the average calculation time, the average total fuel consumption FC_T and its standard deviation σ_{FC} calculated for each method over the 50 initial speed samples.

Method	Niter [-]	Time [s]	$\mathbf{FC_T}$ [kg]	$\sigma_{\mathbf{FC}}$ [kg]
PM	14.0	18.2	466.98	0.002
GPM	6.84	3.24	467.05	0.011
FDM	17.14	8.60	476.34	4.88
SLQP	100.54	33.34	467.80	1.28

Table 3: Comparison of the optimization algorithm performance

From Table 3 it can be seen that each method converges to the same optimum fuel consumption value ($FC_T \approx 467\ kg$) except for the Feasible Direction Method. The standard deviation calculated for FDM and SLSQP is also important which indicates a large spreading of the optimum values found around the mean. Figure 7 illustrates the convergence of the Penalty Method, Feasible Direction Method and Gradient Projection Method for the first point of the random sample.

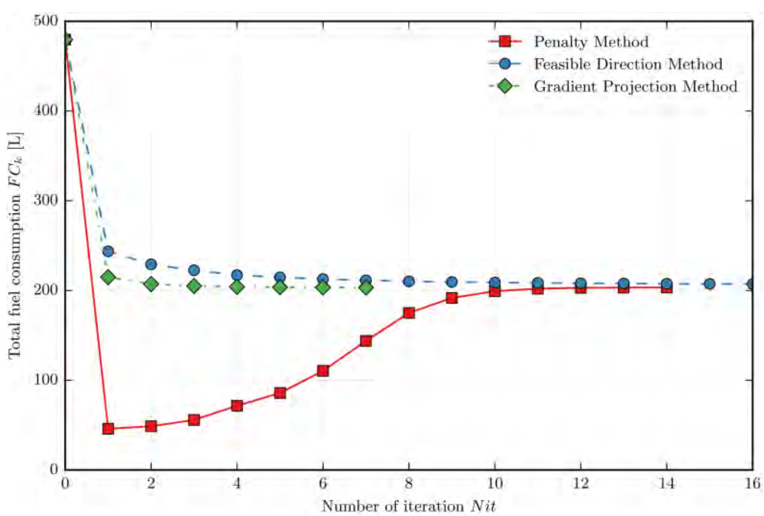


Figure 7 : Illustration of the convergence of Penalty Method, Feasible Direction Method and Gradient Projection Method for a random initial speed

Figure 7 shows that the Gradient Projection Method converges much faster (7 iterations) than the Penalty (14 iterations) and Feasible Direction Method (17 iterations). It can be seen that after the first iteration of the Penalty method, the solution is located outside of the feasible domain (the updated calculated speed solution is small which explains the low fuel consumption FC_T), therefore the penalty function is switched to exterior penalty function, and after a few iterations, the solution is brought back into the feasible domain and converges to the solution.

Table 3 also shows that the Penalty Method and Gradient Projection Method have lower standard deviation σ (better accuracy) and also converge faster than the SLSQP method. In average, the Gradient Projection Method is 6 times faster than the Penalty Method and shows good accuracy (low standard deviation). Overall, the Gradient Projection Method performs better than the three other optimization techniques tested for this problem. This method is particularly adapted for this problem because it projects the search direction into the subspace tangent to the active constraints which is where the solution lies. For these reasons, this method is used to solve the speed optimization problem.

4.3 Application of the speed optimization

Average AIS speed observed on each leg of the travel is used as initial speed profile for the optimization process. AIS data for a 135 m long vessel and covering three full months (November and December 2017 and January 2017) has been used for this study. To be as accurate as possible, for each AIS speed collected, its timestamp has been compared to the measured discharge at Chatou's dam and only AIS data corresponding to the studied discharges have been selected for the calculation of the mean speed on each leg. The discharge measured during this period does not exceed 700 m^3/s. Therefore, speed optimization has been carried out for two river discharges: 200 m^3/s and 500 m^3/s. The maximum travel duration T_{max} is set as the travel time necessary for the ship to complete the itinerary with the mean AIS speed calculated on each leg. The ship draft T is 2 m corresponding to 3 layers of containers used for 80% of the travels for this ship.

Figure 8 and Figure 9 show the profile of (a) instant fuel consumption, (b) speed V and (c) water depth restriction H/T, in the case of AIS speed profile and optimized speed profile for a discharge $Q = 200\ m^3/s$ and a ship sailing upstream (Figure 8) and downstream (Figure 9).

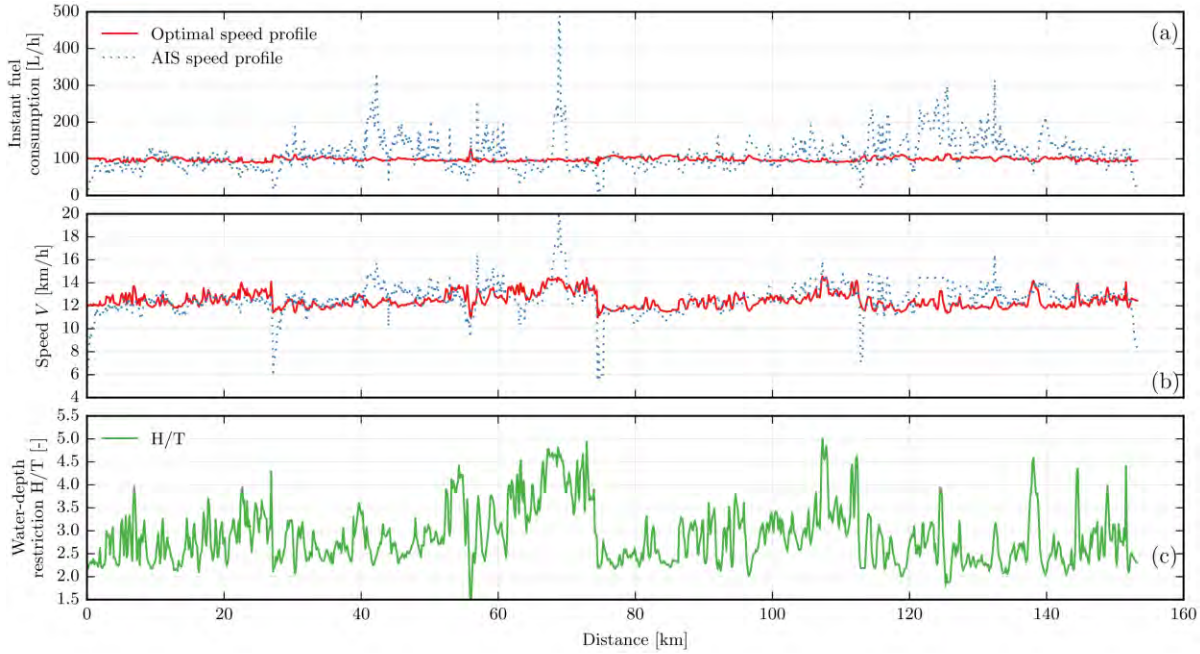


Figure 8: Profile of (a) instant fuel consumption,(b) speed V and (c) water depth restriction H/T for AIS and optimal speed ($Q = 200\ m^3/s$, upstream)

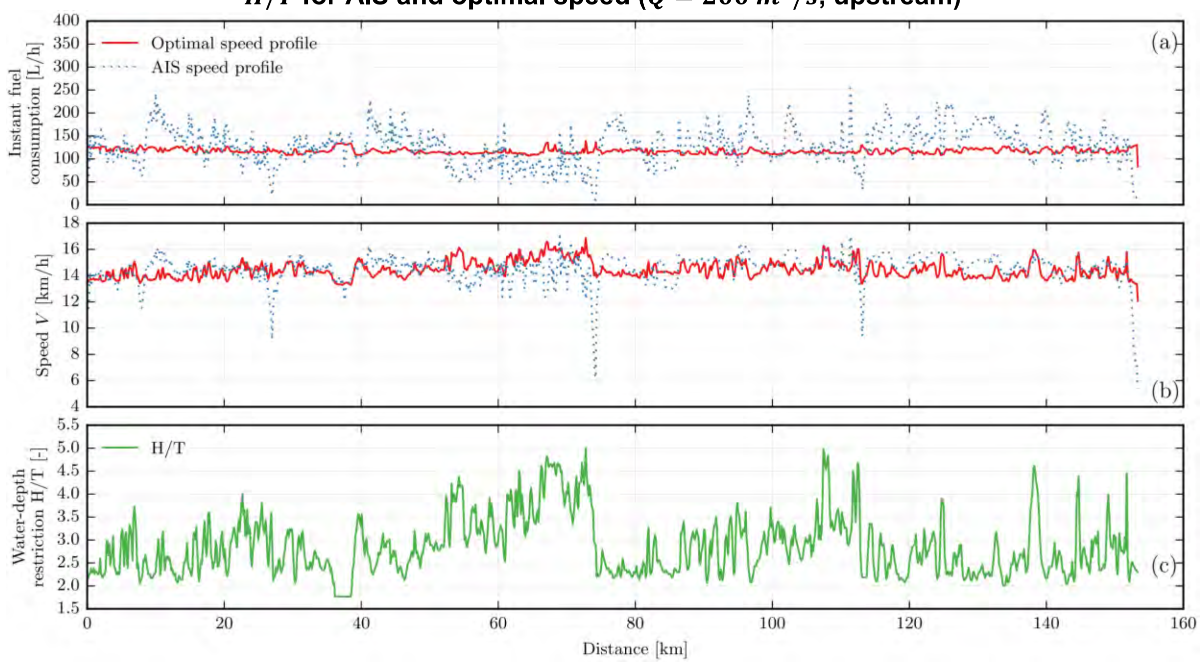


Figure 9: Profile of (a) instant fuel consumption,(b) speed V and (c) water depth restriction H/T for AIS and optimal speed ($Q = 200\ m^3/s$, downstream)

Figure 8 and Figure 9 show that the speed profile observed with AIS data varies significantly over the travel length. The four low speed peaks observed at around 25, 75, 110 and 155 km correspond to a slow down of the ship when approaching a lock on the itinerary. Those two figures also show that the speed variation amplitude is less significant for the optimum speed profile than for the AIS speed profile.

For the optimal speed profiles, the instant consumption is relatively constant over the travel, which is not the case for the AIS speed profile. In those cases, the specific fuel consumption SFC remains nearly constant because the engine is operated at a regime where SFC curve is flat (see Figure 4). As a result, the engine power variation remains limited during the travel, which is in agreement with findings reported by Bons et al. [2014] that minimum fuel consumption is achieved on a waterway by operating in constant power.

The optimal speed profile and H/T profile also have the same shape (Figure 8 and Figure 9). The Seine being a wide river (around a hundred meters), water depth is the main parameter having an effect on the added resistance due to restriction. As a result, the ship will sail faster when restriction is less important and slow down for lower values of H/T.

Table 4 shows the average water depth $\overline{H}$, the average current velocity $\overline{Uc}$, the total fuel consumption for the AIS speed profile FC_{T0} and optimal speed profile FC_{Topt}, the mean fuel consumption (in L/km) for the AIS speed profile $\overline{FC_{T0}}$ and optimal speed profile $\overline{FC_{Topt}}$; and the fuel consumption reduction $FCR = (FC_{T0} - FC_{Topt})/FC_{T0} \times 100$ calculated for each of the 4 cases studied.

Q [m ^{3}/s]	**Dir.** [-]	$\overline{\mathbf{H}}$ [m]	$\overline{Uc}$ [m/s]	$\mathbf{FC_{T0}}$ [L]	$\mathbf{FC_{Topt}}$ [L]	$\mathbf{\Delta FC_T}$ [L]	$\overline{\mathbf{FC_{T0}}}$ [L/km]	$\overline{\mathbf{FC_{Topt}}}$ [L/km]	$\overline{\mathbf{FCR}}$ [%]
200	up	5.7	0.21	1327	1215	112	8.7	7.9	9.3
200	down	5.7	0.21	1335	1244	92	8.7	8.1	7.4
500	up	6.0	0.51	1974	1851	123	12.9	12.0	6.6
500	down	6.0	0.51	674	623	51	4.4	4.1	8.2

Table 4: Speed optimization results calculated for two discharges ($Q = 200\ and\ 500\ m^3/s$) and two sailing directions (upstream and downstream)

The fuel saving for a travel ΔFC_T vary between 51 L and 123 L with an average of 95 L (Table 4). The associated fuel consumption reduction FCR varies between 6.6% and 9.3% with an average of 7.9%. Finally, the average fuel consumption FC_T for the 4 tested cases is equal to 8.7 L/km for the AIS speed profile and 8.05 L/km for the optimum speed profile which is in agreement with the average fuel consumption of 8 L/km indicated by VNF [2006] for this type of vessel. The fuel consumption reduction FCR results presented in the table also show that the fuel savings obtained are more significant in the case of a ship sailing downstream. This difference could be explained by the fact that sailing against the flow limits the possible change in speed as it requires more power to increase the velocity of the ship.

4.4 Influence of trajectory and travel duration

With the aim of studying the influence of the lateral position of the ship in the channel, the previous simulation is compared with the results obtained when the ship is sailing in the deepest part of the river. In the latter scenario, the turning circle of the vessel is not taken into account and choosing the deepest part of the river occasionally create discontinuities in the trajectory. Figure 9 shows (a) the instant consumption, (b) the speed profile and (c) water depth restriction ratio H/T profile against the distance for the two scenarios.

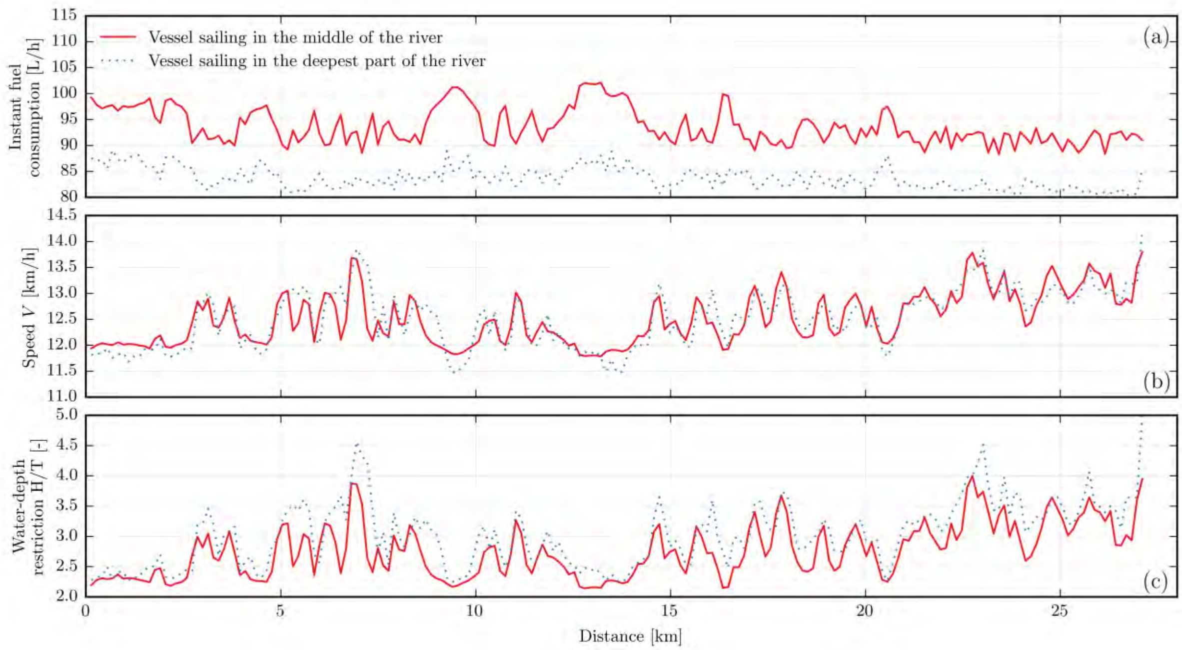


Figure 10: Profile of (a) instant fuel consumption, (b) speed V and (c) water depth restriction H/T for the vessel sailing in the middle and in the deepest part of the river (ship sailing upstream on Reach one for a discharge of 200 m^3/s)

Figure 10 shows that the water depth profile in the case of a ship sailing in the middle and in the deepest part of the river have the same shape; however, the water depth restriction is less important for the ship sailing in the deepest part of the waterway. Both speed profiles also show the same pattern but the ship navigating in the deepest part of the river tends to go faster when the restriction is low and slower when it is important. As a result, the instant consumption is clearly lower in the case of the ship sailing at maximum depth. The main reason for that is that the added force due to water depth restriction is less important when the ship sails in the deepest part of the river. In the case of the ship sailing at maximum water depth, the total consumption obtained is FCTopt = 171:67kg for the optimal speed profile. The comparison with the consumption obtained for the first scenario gives a 9% reduction of the total fuel consumption. Although this reduction could be less important when taking turning circle into account, this result indicates choosing the optimal track can also lead to additional fuel savings. Theoretically, this track could be determined with up to date bathymetry data of Seine river bottom, but other factors also have to be taken into account such as the continuity of the trajectory and locally specific navigation rules.

The influence of travel duration on the optimal fuel consumption has been studied by running several simulations in which the maximum travel duration Tmax is increased, from 2h to 3h30. Figure 10 presents the evolution of the optimal fuel consumption FCT and the fuel consumption reduction defined by $FC_{Red} = \frac{FC_T(2h) - FC_T(T_{max})}{FC_T(2h)}$ against the máximum travel duration T_{max}.

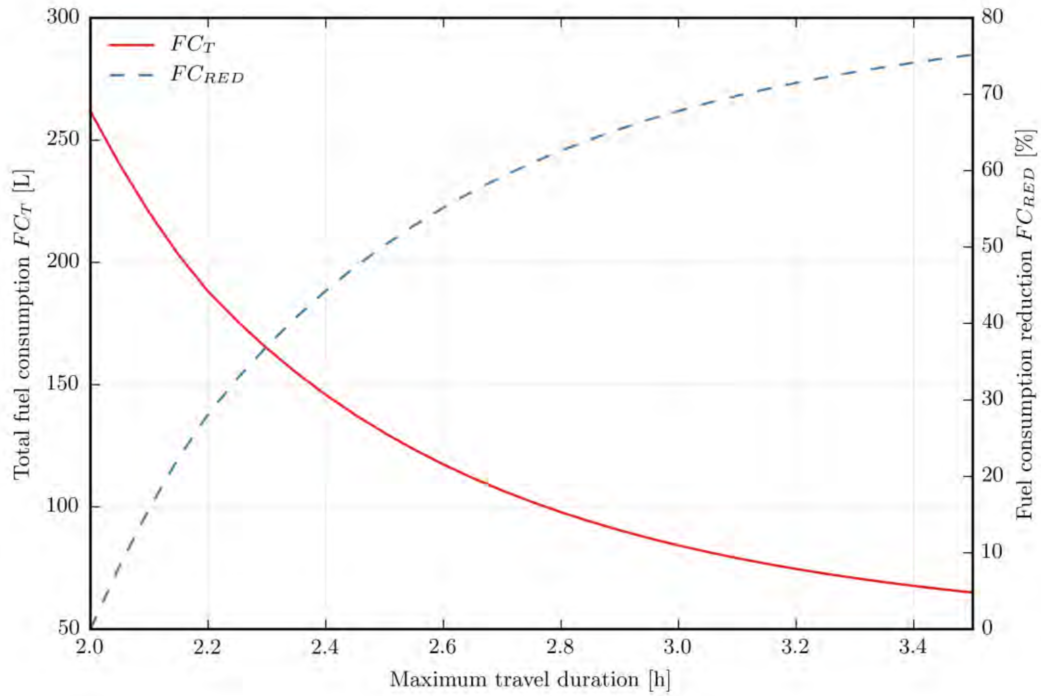


Figure 11: Evolution of total fuel consumption FCT and fuel consumption reduction FCRED against the maximum travel duration Tmax (ship sailing upstream on Reach one for a discharge of 200 m3/s)

The fuel consumption decreases sharply for a travel duration between 2h and 2.5h and at a steadier rate afterwards (Fig 10). The main reason for this evolution is that the thrust power necessary to maintain a ship at a constant speed V is roughly proportional to V^3. Therefore, increasing the maximum travel duration allows to reduce the average speed; and the fuel consumption, linked to the thrust power, decreases in an exponential manner. For instance, a 12 minutes travel time increase, from 2h to 2h12, leads to a 26% fuel consumption decrease. This fact highlights another important aspect of fuel consumption optimization: including real time information in the optimization process can lead to additional fuel savings. For instance, knowing in advance that an approaching lock is unavailable due to maintenance or ship queue can be used to reduce the sailing speed in order to avoid waiting at the lock and make fuel savings.

5. **Conclusions and perspective**

EcoNav and its modules have been described in this paper and applied to a real case study. This model is based on an optimization algorithm minimizing the fuel consumption by finding the optimal speed profile for a given itinerary (operating conditions) under specified constraints (maximum travel duration). The fuel consumption is evaluated with a specific fuel consumption empirical model developed by Hidouche et al [2015] and a ship resistance surrogate model based on ship resistance experimental results from Anast towing tank. The operating conditions used by EcoNav (channel width, water depth and current velocity) are calculated by using a 2D hydraulic model (Telemac2D). Several methods for the surrogate model and the optimization process were tested and allowed to select the most appropriate (in terms of accuracy and speed) for EcoNav. EcoNav has been applied to study the itinerary of the self-propelled ship Oural on river Seine, between the locks of Chatou and Poses. The comparison between the optimum fuel consumption and the mean AIS observed speed on each leg of the itinerary, showed an average calculated fuel savings of 7.9 %. The average calculated fuel consumptions on this itinerary are in agreement with the results reported in VNF [2006]. The comparison of optimal fuel consumption obtained in the case of a ship sailing in the middle or in the deepest part of the waterway demonstrated that significant fuel savings can be expected by optimizing the vessels trajectory. Finally, it was shown that additional fuel consumption reduction can be realized by extending the duration of the travel. The latter solution could be used in case of lock unavailability or heavy traffic and would necessitate including real time information in the optimizing process. Altogether, this paper presented three ways of reducing fuel consumption: optimizing the speed and trajectory of a vessel and including real time information in the optimization process. The speed optimization model presented in this article is still at an early stage design and needs further improvement and validation. Several limitations to this model can be listed:

- the change in the ship speed is instantaneous and the acceleration/deceleration is not taken into account nor its impact on fuel consumption;
- the trajectory is linearised, as a result its curvature and impact on fuel consumption is neglected;
- wind effect on ship resistance and fuel consumption is not taken into account;
- the hydrodynamic model cannot currently simulate a sea tide and therefore EcoNav cannot be used in intertidal area;
- the ship consumption model needs further validation.

This model could be improved by taking into account the acceleration/deceleration of the ship in the fuel consumption model and the optimization process, and including the effects of the trajectory curvature in the model (rudder effects for instance). Existing empirical models for air resistance calculation could also be used to take the effect of wind into account in the fuel consumption model. The accuracy of the propulsion modelling could also be improved by using existing empirical models to calculate the various propulsion efficiencies. A project is currently ongoing to instrumentalize the 135 m self propelled ship Bosphore, in collaboration with Compagnie Fluviale de Transport, in order to realize a broad range of in situ measurement (fuel consumption, engines rpm, ship's speed and position,...) over the span of one year. Data recorded from this project will help improve and validate the ship fuel consumption model. Cerema is also involved in Seine RIS (River Information Service) and will work on the development of three modules: ETA prediction for inland ships, optimization of waiting times in river locks and estuary hydrodynamic model for the prediction of bridge clearance. The feedbacks from this project can contribute to include a sea tide effect and real time traffic information into EcoNav. When EcoNav has reached a higher maturity level, a prototype could be built and tested on a ship in situ.

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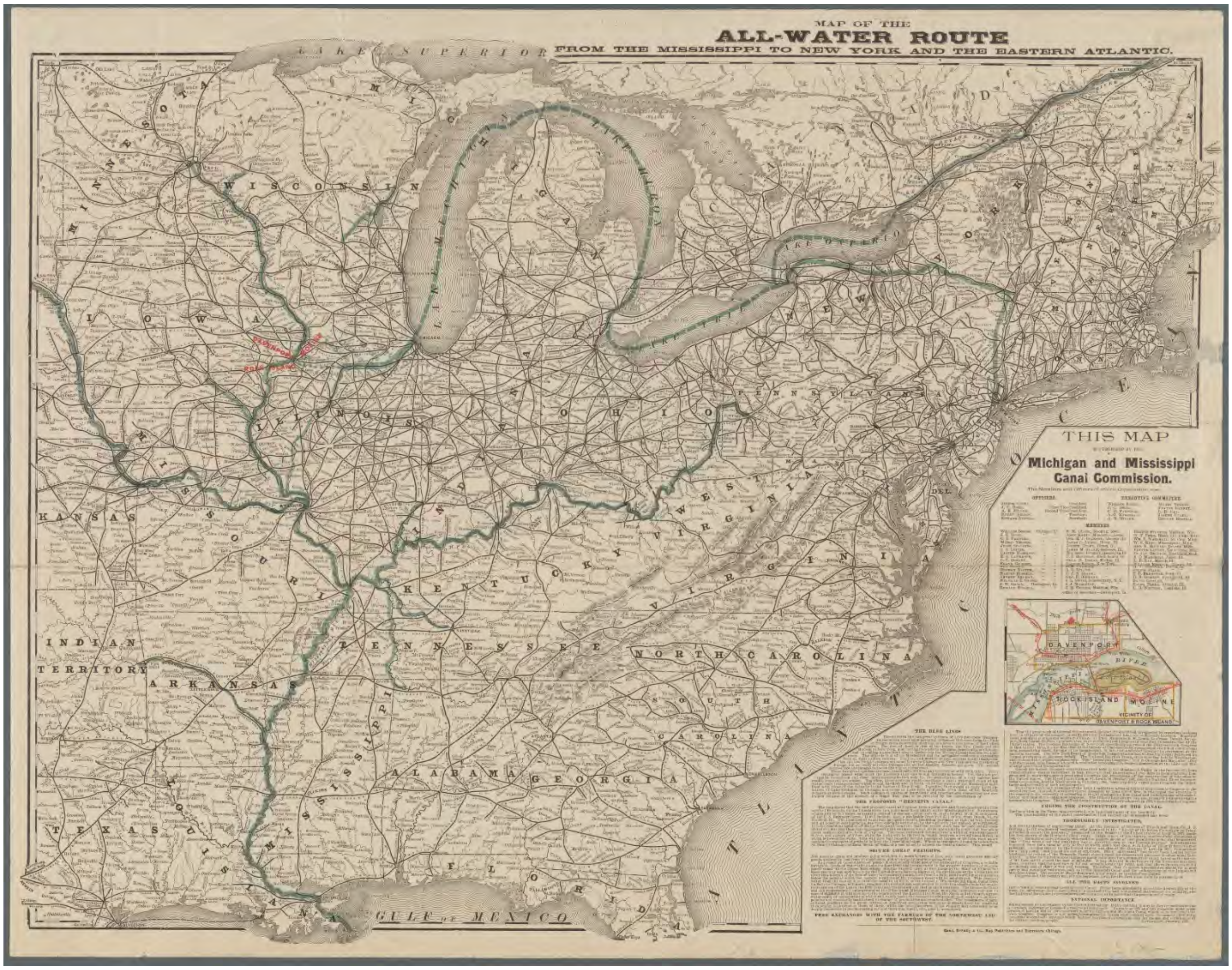

U.S. Waterways: Toward a More Formal Classification in Support of Navigation

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34th PIANC World Congress
June 2018

"All-Water Route" Map Citation: By Lionel Pincus and Princess Firyal Map Division - Lionel Pincus and Princess Firyal Map Division, The New York Public Library. "Map of the all-water route from the Mississippi to New York and the eastern Atlantic" The New York Public Library Digital Collections. 1885. https://digitalcollections.nypl.org/items/f8cb39b0-c76f-0133-12ee-00505686a51c, Public Domain, https://commons.wikimedia.org/w/index.php?curid=60807497

U.S. Waterways: Toward a More Formal Classification in Support of Navigation

Presentation to the 34th PIANC World Congress

CITATION: Brohl, Helen; U.S. Waterways: Toward a More Formal Classification in Support of Navigation, Washington, DC, Paper to the 34th PIANC World Congress, June 2018.

The observations expressed in this paper do not necessarily represent the opinion of the U.S. Committee on the Marine Transportation System

Acknowledgement to Dr. Sandra Knight, Mr. Jeff McKee, and Mr. Robert Dave Lewald for their insights and expertise.

INTRODUCTION:

In 2017, representative members from the U.S. Section of PIANC joined the PIANC Working Group 201 on the development of a proposal of inland waterway classification for South America. All working group members were invited to complete a survey on current classification practices for their respective countries. Interestingly, the three U.S. respondents replied differently to the question of whether the U.S. had a formal classification system. This observation raised the question to the author about the nature of any classification or "non-classification" of U.S. waters and whether navigation interests would benefit from a more formal designation of U.S. waterways. For purposes of this report, waterways includes both coastal and inland systems.

The United States (U.S.) has the largest and most extensive freight infrastructure of any country in the world, comprising over 4.2 million miles of roads, nearly 140,000 miles of railroad lines, and 2.7 million miles of oil and gas pipelines.[1] The United States (U.S.) boasts 12,000 miles of coastline[2] including 25,000 miles of commercially navigable harbors, channels, and inland waterways[3], as well as 241 locks at 195 locations.[4] In 2015, vessels carried $1.6 trillion in imports to and exports from the United States. Water is the leading transportation mode for U.S.-international trade both in terms of weight and value. Ships move more than 71.1 percent of trade weight and 41.8 percent of trade value.[5] Domestic tonnage for coastwise, lakewise, and internal movements totaled 1.7 trillion short tons in 2016.[6] More domestic tonnage moved on the inland rivers than coastal or in the Great Lakes but the longest average haul was about twice as long via coastal routes.[7]

The U.S. Marine Transportation System (MTS) is the waterborne-related component of the U.S. National Freight System. The MTS is made up of an array of interdependent parts, including ports, terminals, vessels, intermodal connectors and the companies and organizations that use them. In addition to freight, the MTS supports the movement of passengers by ferries and cruise ships, commercial and recreational fishing, military cargoes and recreational boating. The navigable coastal, Great Lakes, and inland systems are supported by both physical and informational infrastructure, including canals, locks, dams, aids to navigation (ATON), and deep and shallow draft channels and harbors.

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[1] U.S. Department of Transportation, Bureau of Transportation Statistics, Transportation Statistics Annual Report 2016 (Washington, DC: 2016), pg v.

[2] CIA World Factbook: United States. Available at https://www.cia.gov/library/publications/resources/the-world-factbook/geos/us.html, April 2018.

[3] U.S. Department of Transportation, Bureau of Transportation Statistics. 2016 Pocket Guide to Transportation. Available at: http://www.rita.dot.gov/bts/sites/rita.dot.gov.bts/files/Pocket%20Guide%202016.pdf, May 2016.

[4] U.S. Department of Defense, US Army Corps of Engineers, Navigation Data Center, U.S. Waterway System: Transportation Facts and Information (revised June 2015). Available at: http://www.navigationanddatacenter.us/factcard/factcard14pdf.

[5] Ibid, pg 74.

[6] U.S. Department of Defense, U.S. Army Corps of Engineers, 2016 Fact Card, Available at http://www.navigationdatacenter.us/factcard/FactCard2016.pdf, 2016.

[7] Ibid.

Jurisdictions within the U.S. MTS are complex and decentralized, consisting of federal, state, local, and privately-held components. Within the Federal Government alone, over 30 agencies, bureaus, White House offices, and Federal interagency organizations ranging from the U.S. Coast Guard (USCG) and the U.S. Army Corps of Engineers (USACE) to the National Oceanic and Atmospheric Administration (NOAA), and Maritime Administration, to name a few, are involved in some aspect of MTS oversight, support or engagement.[8]

Only Congressionally authorized navigable channels can be maintained by the U.S. government. These federal navigation channels are approved, individually, in statute for construction, operation and maintenance. Construction of coastal harbors and channels are funded by federal construction appropriations and non-Federal sponsors. Construction and major rehabilitation of inland waterways, including locks and dams are funded by federal construction appropriations and the Inland Waterways Trust Fund. Operations and maintenance funding for deep-draft waterways is appropriated annually and allocated based on use, available funding, and hydrologic conditions through the Harbor Maintenance Trust Fund. Maintenance of locks and dams (USACE), nautical charts (NOAA), and ATONS (USCG), for example are funded by yearly appropriations directly to the agency of oversight.

Therefore, not only is the U.S. marine transportation system vast, complex, decentralized with many components and funding mechanisms, there is no single source to classify the 25,000 miles of navigable waters. The question posed for this paper is to what extent does the U.S. currently classify its waterways, and is there a mechanism(s) that would enhance the operations and efficiency requirements and use of U.S. coastal and inland waterways.

BACKGROUND:

Within the geographic vastness of the U.S., resides eleven domestic and transboundary large basins: Great Lakes Seaway System; Ohio River Basin; Delaware River Basin; Illinois River; Kentucky River; Mississippi River Basin; Missouri River Basin; Columbia River Basin; Gulf Intracoastal Waterway; Atlantic Intracoastal Waterway; and the Tennessee-Tombigbee System. These basins further include large waterway systems such as the Cumberland River and McClellan Kerr Arkansas River. As noted, the United States (U.S.) boasts within these large basins, 12,000 miles of coastline[9] including 25,000 miles of commercially navigable harbors, channels, and inland waterways[10], including 241 locks at 195 locations.[11]

The basis for federal jurisdiction over navigable waters in the United States (U.S.) lies in the Commerce Clause of the U.S. Constitution. Since the early

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[8] U.S. Committee on the Marine Transportation System, Compendium of Federal Programs in the Marine Transportation System, Available at https://www.cmts.gov/resources/compendium, Washington, DC, 2010.

[9] CIA World Factbook: United States. Available at https://www.cia.gov/library/publications/resources/the-world-factbook/geos/us.html, April 2018.

[10] U.S. Department of Transportation, Bureau of Transportation Statistics. 2016 Pocket Guide to Transportation. Available at: http://www.rita.dot.gov/bts/sites/rita.dot.gov.bts/files/Pocket%20Guide%202016.pdf, May 2016.

[11] U.S. Department of Defense, US Army Corps of Engineers, Navigation Data Center, U.S. Waterway System: Transportation Facts and Information (revised June 2015). Available at: http://www.navigationanddatacenter.us/factcard/factcard14pdf.

nineteenth century, the U.S. Supreme Court has held that the Commerce Clause[12] gives the federal government extensive authority to regulate interstate commerce. This view originated in an 1824 case of (GIBBONS V. OGDEN), where the Court was faced with deciding whether to give precedence to a state or Federal government for the licensing of vessels. The Court ruled that navigation of vessels in and out of ports of the nation is a form of interstate commerce, giving Federal law precedence and subsequently leading to broad Federal power over navigable waters.

The term "navigable waters of the U.S." was derived from the Rivers and Harbors Act of 1899 to identify waters that were involved in interstate commerce and designated as federally protected waters. Since then, court cases have further defined navigable waters to include waters that are not traditionally navigable. [13] This is of importance in the U.S. because it impacts jurisdiction, the guiding mechanism to implement permitting and oversight requirements.

There is a vast body of Federal regulation concerning navigable waters, a few of which are listed in Appendix 1. A definition of waters of the United States as it relates to the U.S. Army Corps of Engineers (USACE) can be found under Title 33, Part 328 of the U.S. Federal Code of Regulations (CFR) to include all waters which are "currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters subject to ebb and flow of the tide."[14] This is a very broad and encompassing descriptive which does not further categorize the type and size of vessel required or allowed for navigation.

Further, a waterway may be defined as navigable even if only traversed by canoes. As the result of a 1979 case (Kaiser Aetna v United States), the U.S. Supreme Court established four tests for determining what constitutes navigable waters:

1. Is the body subject to the ebb and flow of the tide;
2. Does the body of water connect with a continuous interstate waterway;
3. Does it have navigable capacity; and
4. Is the body of water actually navigable[15]

By comparison, the European Conference of Ministers of Transport 1972 Resolution No. 92/2 on "New Classification of Inland Waterways," inland waterways are classed by:

- Regional and international importance;
- Type of vessel (length, beam, draught, tonnage);
- Type of convoy (length, beam, draught, tonnage); and

[12] The Commerce Clause provision of the U.S. Constitution gives Congress exclusive power over trade activities among the states and with foreign countries and Indian tribes.
[13] National Association of Counties. "Policy Brief." 2014. http://www.naco.org/sites/default/files/documents/Waters-of-the-US-County-Analysis.pdf
[14] U.S. Government Publishing Office. Electronic Code of Federal Regulations. https://www.ecfr.gov/cgi-bin/text-idx?SID=49de504bf987294d7baa5ab91698c81a&mc=true&node=se33.3.328_13&rgn=div8 (October 26, 2017).
[15] Free Dictionary, "Navigable Waters," 2018, Fairlex, Inc. https://legal-dictionary.thefreedictionary.com/Navigable+Waters.

- Minimum clearance under bridges.[16]

WATERS OF THE U.S.

Navigability is clearly a primary classification of a U.S. waterway, and remains so even if the waterway is no longer navigable.

The terms "deep draft" and "inland" are commonly used to describe waters that can be traversed by ships versus barges, respectively. A deep-draft harbor is defined in law (Title 33 USCS 2241 (1)) to mean a "harbor which is authorized to be constructed to a depth of more than 45 feet (13.7 meters) and in some cases to 53 feet. "Authorized" simply means that, if the funding and funding mechanisms were in place, a harbor could be deepened to 45 feet. Hence, not all harbors and channels are authorized to a greater depth and not all authorized harbors and channels have the funding and funding mechanisms in place to meet the deep-draft definition. The Great Lakes is often referred to as being "deep draft" even though the operating depth is 26.6 feet (8 meters).

U.S. Coast Guard regulations under Title 33, Section 2.26 of the CFR, defines inland waters to mean the waters shoreward of the territorial sea baseline. Coast Guard regulations also define waters for territorial seas, internal waters, contiguous zone, exclusive economic zone, and high seas. The inland waterways system comprises navigable rivers linked by a series of major canals. Lock and dam infrastructure is the chief mechanism in enabling the upstream and downstream movement of cargo.

Interestingly, Title 33, Part 329.4 also provides a definition of "navigable waters of the United States" almost identical to that under Part 328, except to also add that "a determination of navigability, once made, applies laterally over the entire surface of the waterbody, and is not extinguished by later actions or events which impede or destroy navigable capacity." [17] Though navigability is clearly a primary classification of a U.S. waterway, the availability of operational capabilities is not guaranteed. In other words, once a waterway is classified as navigable, it remains so, even if it is no longer navigable for one reason or another.

The US Army Corps of Engineers estimated that full channel dimensions at the nation's busiest authorized channels were available less than 35% of the time.[18] Increased maintenance funding and supplemental funding (post extreme weather events) since that 2011 analysis has contributed to improved operational capabilities but improvements are not addressed to provide a whole-of-nation condition. This is a function of current legal authorities and funding mechanisms.

Rivers are also formally "classified" as wild, scenic or recreational under the National Wild and Scenic Rivers Act of 1968, but do not provide a commercial navigation perspective. This law sought to preserve certain rivers with outstanding natural, cultural, and recreational values in a free-flowing condition for the enjoyment of present and future generations. The goal was to safeguard the special character of a river, while recognizing the potential for appropriate use

[16] International Transport Forum, ECMT Resolution No. 92 on New Classification of Inland Waterways, 1972, Paris, France; https://www.itf-oecd.org/sites/default/files/docs/wat19922e.pdf.
[17] U.S. Government Publishing Office. Electronic Code of Federal Regulations. https://www.ecfr.gov/cgi-bin/text-idx?SID=5344d2287fc4259450ad48e96fe815d0&mc=true&node=se33.3.329_14&rgn=div8 (October5 26, 2017)
[18] Cater, Nicole; Stern, Charles, Congressional Research Service, Army Corps Fiscal Challenges: Frequently Asked Questions, (7-5700), December 15, 2011, Washington, DC.

and development as well as encouraging river management across political boundaries. Rather, the goal is to balance the location of dams and development against natural, free flowing river aspects.[19]

While there are extensive references in U.S. history to the use of waterways for navigation, much of the river systems were first altered in an effort to control flooding or to drain adjacent wetlands for agricultural purposes. Subsequent flood-control levies and dams, and locks then established the operating controls for those rivers. The USACE Navigation Data Center reports that the average control depth for deep draft navigation is 35 feet, 10 feet for shallow draft navigation and 28 feet for both deep and shallow draft.[20] But, again, these are system-wide averages requiring navigation interests to review specific areas of interest.

In 2010, the U.S. Department of Transportation identified 18 “marine corridors” and 6 initiatives for further development as part of “America’s Marine Highway Program.” [21] Directed in statute, the Department designated “short sea transportation routes” as extensions of the surface transportation system to focus public and private efforts to use the waterways to relieve landside congestion along coastal corridors. However, these marine highway corridor designations were not intended to classify waterways for navigation purposes. Rather, a corridor designation provides a framework under which business development proposals within a corridor could be submitted to the Department when Congress provided the funds.

OBSERVATIONS

The U.S. has clear definitions of “navigable” waters and Federal jurisdictions. The operational boundaries of navigable waters are defined by existing control infrastructure such as locks, dams, and levies or maintained depths. While there are national averages for deep draft and shallow draft waterways, the operational capabilities of a given waterway must be explored individually. The breadth, scope, and type of navigable waters in the U.S. make it impossible to classify or categorize vessel operations across all basins.

For example, the Missouri River is one of the largest rivers in North America. The Missouri River originates in southwestern Montana and flows in a southeasterly direction about 2,315 miles to join the Mississippi River above St. Louis, Missouri. However, navigation on the river only occurs between Sioux City, Iowa to St. Louis, a distance of 734 miles with a 9-foot deep, 300-foot wide navigation channel.[22] Because the river is large, it is subject to wind and wave action, and allows mixed use to include recreational boating, fishing and kayaking. Navigation charts are provided by the U.S. Army Corps of Engineers and ATONS are maintained by the U.S. Coast Guard.

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[19] U.S. Department of Interior, National Wild and Scenic Rivers System. https://www.rivers.gov/wsr-act.php. October 30, 2017.

[20] USACE Navigation Data Center GIS Viewer (http://www.navigationdatacenter.us/db/gisviewer, file ndcgis13shp.zip, updated May 16, 2013; accessed July 2014 by The National Academy Press and viewed at: https://www.nap.edu/read/21763/chapter/4.

[21] The final rule published by U.S. DOT was in response to a directive in the Energy Independence and Security Act of 2007, enacted December 19, 2007, https://www.gpo.gov/fdsys/pkg/FR-2010-04-09/pdf/2010-7899.pdf.

[22] U.S. Army Corps of Engineers, Omaha District, Missouri River Navigation, http://www.nwo.usace.army.mil/Missions/Dam-and-Lake-Projects/Missouri-River-Navigation/, April 2018.

Vessel access into the Great Lakes is limited by the lock system on the Saint Lawrence River which opened in 1959. There are 13 Canadian-owned locks and two U.S.-owned locks that limit vessel size to 740 feet (227.7 meters) in length, 78 feet (23.8 meters) in width and 26.6 feet (8 meters) in depth to transit.[23] The U.S. and Canadian Seaway authorities manage all traffic control through that system for which navigation information is provided in a regularly updated handbook. In many respects, this portion of the Seaway has a classification similar to the European model providing for ship transit procedures and practices. However, transiting into ports within the Great Lakes requires knowledge of in-transit river systems and specific harbor dimensions. Nautical charts for the system are provided by the Canadian Hydrographic Service and the U.S. National Oceanic and Atmospheric Administration.

The man-made canals in the U.S. may better parallel the European model. For example, the Gulf Intracoastal Waterway (GIWW) is a 1,300 mile, man-made canal, authorized to 12 feet, that runs along the Gulf of Mexico coastline from Brownsville, Texas to St. Marks, Florida. There are 54 bridges along its stretch but only one of them is considered a bottleneck with minor delays near New Orleans.[24] The Gulf Intracoastal Canal Association provides an alert service with near real-time reports on navigation issues affecting waterway traffic through a third-party provider, but official nautical charts are provided by the National Oceanic and Atmospheric Administration in, what is termed, "small craft charts." Pilotage guides are available from the private sector, however, there is no GIWW "handbook" for the system, similar to what is provided by the Seaway managers. Though it's an open public waterway, commercial operators are domestically licensed and, generally, have familiarity with the waters and operational limitations. As a result, the need for formal operational classification may not be critical.

The Tennessee-Tombigbee Waterway (Tenn-Tom) is a large national public works project by the US Army Corps of Engineers that opened for commerce in January 1985, to more efficiently link the Tennessee and Tombigbee Rivers southward to Mobile, Alabama. While lock operations on the Tenn-Tom are managed by the USACE, there is a waterway development organization (tenntom.org) that readily provides operational classifications; i.e. 234 miles long, minimum depth of 9 feet, 10 locks handling an 8-barge tow and prescribed bridge clearances. Navigation charts are provided by the US Army Corps of Engineers, Mobile District, supplemented by Notices to Navigation Interests. In fact, USACE provides this type of information for most of the nation's inland system. Again, there is no "handbook" for traversing the waterway but commercial operations require domestically licensed operators with familiarity of the waterway.

How does the United States classify its waterways for navigation? The designation of navigable versus non-navigable is the broadest, accepted descriptor. A second classification is deep-draft or shallow-draft. One can easily make reference of a regional, national or international significance of a given basin, waterway or harbor. The ability of a vessel or convoy of barges to traverse a waterway is very specific to a given waterway and to the extent that the operational limitations of a waterway are provided, that specific waterway is classified similar

[23] The St. Lawrence Seaway Management Corporation, Seaway Handbook 2018 Edition, http://greatlakes-seaway.com/seaway_handbook/table-of-contents-e.html, April 2018.
[24] Gulf Intracoastal Canal Association, conversation with James Stark, Executive Director, April 12, 2018.

to the European model. Notwithstanding that U.S. waterways are not developed or maintained at equal amounts to each other across the nation, a given waterway is likely used effectively and safely. The sheer size and diversity of the U.S. maritime transportation system makes it challenging, but perhaps it is at the policy level that a holistic classification of U.S. waterways would be of benefit.

APPENDIX 1

CITATION	DEFINITION	FEDERAL AGENCY
Title 33, Part 328 CODE OF FEDERAL REGULATIONS (CFR)	All waters which are currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters subject to ebb and flow of the tide.	US Army Corps of Engineers
Title 33, Part 207 CFR	Operational procedures including navigation regulations, flood control, drinking water, removal of wrecks, aquatic plan control, permitting…	US Army Corps of Engineers
Title 33, Part 329.4	Adds "a determination of navigability, once made, applies laterally over the entire surface of the waterbody, and is not extinguished by later actions or events which impede or destroy navigable capacity."	US Army Corps of Engineers
Title 33, Section 2.26	Inland waters are waters shoreward of the territorial sea baseline.	US Coast Guard
Rivers and Harbors Act of 1899 and related court cases	Defined navigable waters of the U.S. even if not traditionally navigable. Forbids building any unauthorized obstruction to the nation's navigable waters and gives enforcement powers to the USACE.	U.S. Federal Government US Army Corps of Engineers
National Wild and Scenic Rivers Act of 1968	Preserve certain rivers with outstanding natural, cultural and recreational values in a free-flowing condition for enjoyment by safeguarding special character of a river to include river management. Rivers are either wild, scenic or recreational but no commercial navigation perspective.	U.S. Federal Government
U.S. Constitution, Article 1, Section 8 (Commerce Clause)	Gives the Federal government extensive authority to regulate	U.S. Federal Government

[Gibbons v Ogden (1824)]	interstate commerce and broad power over navigable waters.	
Longshore and Harbor Workers' Compensation Act of 1988	Provides that employers are liable for injuries to sailors that occur upon navigable waters of the United States.	Department of Labor
Title 40 CFR 230.3(s) Defines "Waters of the United States" https://www.epa.gov/cwa-404/definition-waters-united-states-under-clean-water-act	All waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide; All interstate waters including interstate wetlands; All other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce including any such waters: Which are or could be used by interstate or foreign travelers for recreational or other purposes; or (From which fish or shellfish are or could be taken and sold in interstate or foreign commerce; or Which are used or could be used for industrial purposes by industries in interstate commerce; All impoundments of waters otherwise defined as waters of the United States under this definition; Tributaries of waters identified in paragraphs (s)(1) through (4) of this section; The territorial sea; Wetlands adjacent to waters (other than waters that are	EPA; USACE

	themselves wetlands) identified in paragraphs (s)(1) through (6) of this section; waste treatment systems, including treatment ponds or lagoons designed to meet the requirements of CWA (other than cooling ponds as defined in 40 CFR 423.11(m) which also meet the criteria of this definition) are not waters of the United States.	
Energy Independence and Security Act of 2007	Program to expand the use of our Nation's navigable waterways to relieve landside congestion, reduce air emissions, provide new transportation options, and generate other public benefits by increasing the efficiency of the surface transportation system. The program works with public and private stakeholders to achieve these goals.	Maritime Administration, USDOT
1849 Swamp and Overflowed Land Act	The Act entrusted certain wetlands to the states for the purpose of constructing necessary levees and drains to reclaim the swamp and overflowed land.	N/A

FIBER REINFORCED POLYMER COMPOSITE IMPLEMENTATION IN NAVIGATION STRUCTURES

by

Jonathan C. Trovillion[1], Jeffrey P. Ryan[2], and John W. Harper[3]

ABSTRACT

This paper describes a process for the implementation of fiber reinforced polymer (FRP) composites in U.S. Army Corps of Engineers (USACE) navigation structures. The main driver for this work is to reduce fabrication and long term maintenance costs. Successful demonstration projects including FRP composite miter gate contact blocks and wicket gates are presented. Participation in international collaborative efforts for technology exchange is described. A path forward for implementation in increasingly larger gate structures is given which includes identifying specific locations, design considerations, knowledge gaps, and guidance development.

1. INTRODUCTION

The U.S. Army Corps of Engineers (USACE) maintains 12,000 miles of inland waterways in the United States which contain 200+ inland navigation structures such as locks and dams. Approximately 600 million tons of cargo is moved along these waterways and through these structures annually. The majority of these structures were constructed in the 1930-1950 time period with a 50 year design life. Due to national budgetary constraints, these aging structures are not being replaced, but simply maintained. However, the rate of degradation due to corrosion and wear is exceeding the rate at which they can be maintained. The risk of a prolonged failure of components to these structures, which could have a devastating effect to commerce, is becoming more likely every year. For this reason, USACE has placed a high priority on developing novel materials to repair or replace aging infrastructure in order to extend the structures working lifetime. Emphasis has been placed on cost reductions for both first costs and long term maintenance costs, reductions in labor to implement and maintain, as well as longer durability.

2. SUCCESSFUL DEMONSTRATION PROJECTS

2.1 General

In 2014, USACE researchers began collaborating with West Virginia University (WVU) to develop and deploy fiber reinforced polymer (FRP) composite components and structures to repair or replace deteriorating components in locks and dams. These demonstrations showed that significant cost savings in manufacturing and maintenance could be realized while extending the service life of the structures.

Additionally, these projects allowed USACE researchers to develop and demonstrate design and manufacturing techniques that will be appropriate for future implementation and design of FRP composite navigation structures. Two of the most successful of these demonstrations were the development of FRP composite miter blocks and FRP composite wicket gates.

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2.2 Miter Gate Contact Blocks

USACE and WVU developed an FRP composite contact block to replace carbon steel contact blocks on miter gates at the Auxiliary Lock of the Hiram Chittenden Locks in Seattle, WA. This lock has the highest amount of lockages per year than any other lock in the USACE inventory. According to the USACE Lock Performance Monitoring System (LPMS), the lock performed 52,500+ lockages between March 2013 and March 2018. The purpose of the miter blocks is to transfer the hydraulic load placed on the miter gates to the lock wall as well as provide a water tight seal. These blocks experience an abrasive force upon opening and closing that most coatings cannot withstand. Therefore, a microscopic layer of corrosion forms just at the surface of the miter block and is abraded away during operation; as shown in Figure 1. This section loss over time causes a loss in hydraulic seal resulting in a redistribution of loads and stresses in the gates. The material section loss due to abrasion at the Hiram Chittenden Auxiliary Lock was very pronounced on the miter blocks due the unusually high lockages experienced by the gate as well as the brackish water immersion conditions.

Figure 1: Corroded and Abraded Carbon Steel Miter Blocks

Because FRP composite materials are free of corrosion and have good abrasion resistance, a prototype FRP composite miter block was developed to provide a durable alternative to the corrosion prone steel blocks. As no commercial designs of FRP composite miter blocks were available, the prototype was developed through an iterative design process. The initial design requirements for the FRP blocks were to achieve a Factor of Safety of 3 for a 1400 psi working stress. Through several iterations, it was found that a monolithic block of FRP composite material was optimal. The blocks were fabricated using a thermal press technique. To create the blocks, a large panel was manufactured with a 10 mil surface veil and 96 layers of 24 oz. woven glass rovings of PPG Boron. The panel assembly was then vacuum sealed and cured at 120˚ F for 120 minutes. Once cured, the panel was cut into the desired block sizes which were then feathered and sealed using two coats of FR992 resin and wax additive. For a cross section of the blocks, see Figure 2. Laboratory evaluations of the blocks resulted in a maximum compressive strength of 51 ksi along the mitering surface which was significantly higher than that required.

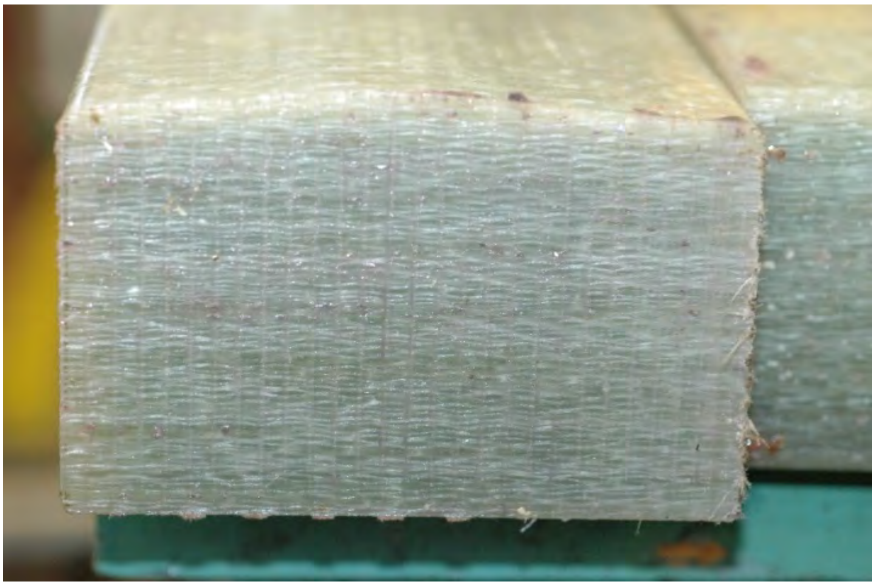

Figure 2: Cross section of FRP Composite Miter Block Prototype

After the compressive strength of the blocks was determined, the FRP miter blocks were installed in the auxiliary lock chamber’s upstream gates. During installation, an additional advantage to the durability and corrosion resistance of the composite blocks was their light weight. Field personal were able to handle the miter blocks by hand for rapid installation and improved safety while steel blocks were heavier and required greater lifting capacity; as shown in Figure 3. The blocks were successfully installed in March 2015 and their performance is being monitored annually. The installation and development of these blocks was significant as it demonstrated that steel components on a larger structure could successfully be replaced with light weight, corrosion resistant composite materials with equivalent or greater mechanical properties.

Figure 3: Field Personnel Handling FRP Miter Blocks

2.3 Wicket Gates

Wicket dams were once prevalent throughout the USACE inventory. Now there are only a few wicket dams left. Two of these are on the Illinois River at the Peoria Lock and Dam and at the LaGrange Lock and Dam. In these dams, wicket gates are used to create an adjustable dam, depending on river conditions. During low water, the wicket gates are raised upright to maintain pool levels for navigation. During high water, the gates are lowered to the bottom, creating a navigable pass which allows river traffic to flow without locking through the lock chamber. Wicket gates are traditionally timber structures. The timber wicket gates located at Peoria Lock and Dam measure 4 feet x 16 feet 5 inch x 12 inch. They are manufactured in-house out of White Oak timber. The timber wicket gates are expensive to manufacture due to the fact that four 12 inch x 12 inch x 20 feet boards are needed for each gate. These timber wicket gates are prone to rot (see Figure 4) with a service life of only 10-20 years in the river environment. Replacing the deteriorated gates is a costly and dangerous process due to the high water flow surrounding the gates. Due to these factors, a FRP composite gate with lower fabrication and maintenance costs and an extended life span was highly desirable.

Figure 4: Deteriorated Timber Wicket Gate

To develop the FRP composite wicket gate, USACE and WVU collaborated with a private industry polymer composite manufacturer, Composite Advantage of Dayton Ohio. There was no engineering design for the current timber wicket gates or design guidance on the expected service loads. The first step was to obtain an example timber wicket gate and reverse engineer it. Calculations were made to determine the capacities of a timber gate and to predict the service loads for various load cases. Since it was critical to maintain the same structural and operational performance, the new composite gates were required to have the same overall dimensions, weight, buoyancy, and center of gravity as the original timber gates. Once these values had been established, an FRP gate was engineered to provide an equivalent performance. Relevant mechanical properties are given in Table 1. In addition, it was decided that the FRP composite gate could be made to be 9 inches thick as opposed to 12 inches for the timber gates. The design bending stiffness of the composite gate was lower than that of the timber gates due to its reduced 9 inch thickness. Had the composite gate been designed with the same 12 inch thickness as the timber gates, its bending stiffness would have been nearly identical to that of the timber gates. It was determined that the lower bending stiffness did not affect operation under service loads.

Mechanical Property	Timber	FRP	FPR/Timber
Allowable Moment (kip-ft)	120	169	1.4
Allowable Shear (kip)	75	137	1.8
Bending Stiffness, EI (lbf-in^2)	6.6x10^9	3.1x10^9	.5

Table 1: Comparison of Timber and FRP Composite Wicket Gate Mechanical Properties

The composite gate was designed as a monolithic sandwich panel consisting of FRP composite faceskin laminates to provide the moment capacity with a bidirectional fiber wrapped closed cell foam core to provide the shear and punching shear strength along both directions of the gate. Vinyl ester resin was selected as the polymer matrix to provide optimum environmental resistance and long term durability in the immersion environment. The glass fiber used to manufacture the laminates and webs of the gate was E-glass. The type of closed cell foam blocks used in the core was polyisocyanurate foam. The foam, used to fill void space and shape the gate, was nonstructural with the entirety of the shear reinforcement being provided by the FRP webs.

The composite wicket gates were manufactured with a vacuum assisted resin transfer molding (VARTM) process, where multiple layers of fabric sequenced to produce the proper fiber architecture for the faceskins were laid into a form. The foam core wrapped with fiber reinforcement members were then placed on top of the faceskin fabric as shown in Figure 5. Once the top faceskin fabric layers were placed to encapsulate the fabric wrapped foam core, the gate was sealed with a vacuum bag and infused with vinyl ester resin to fully saturate the fibers.

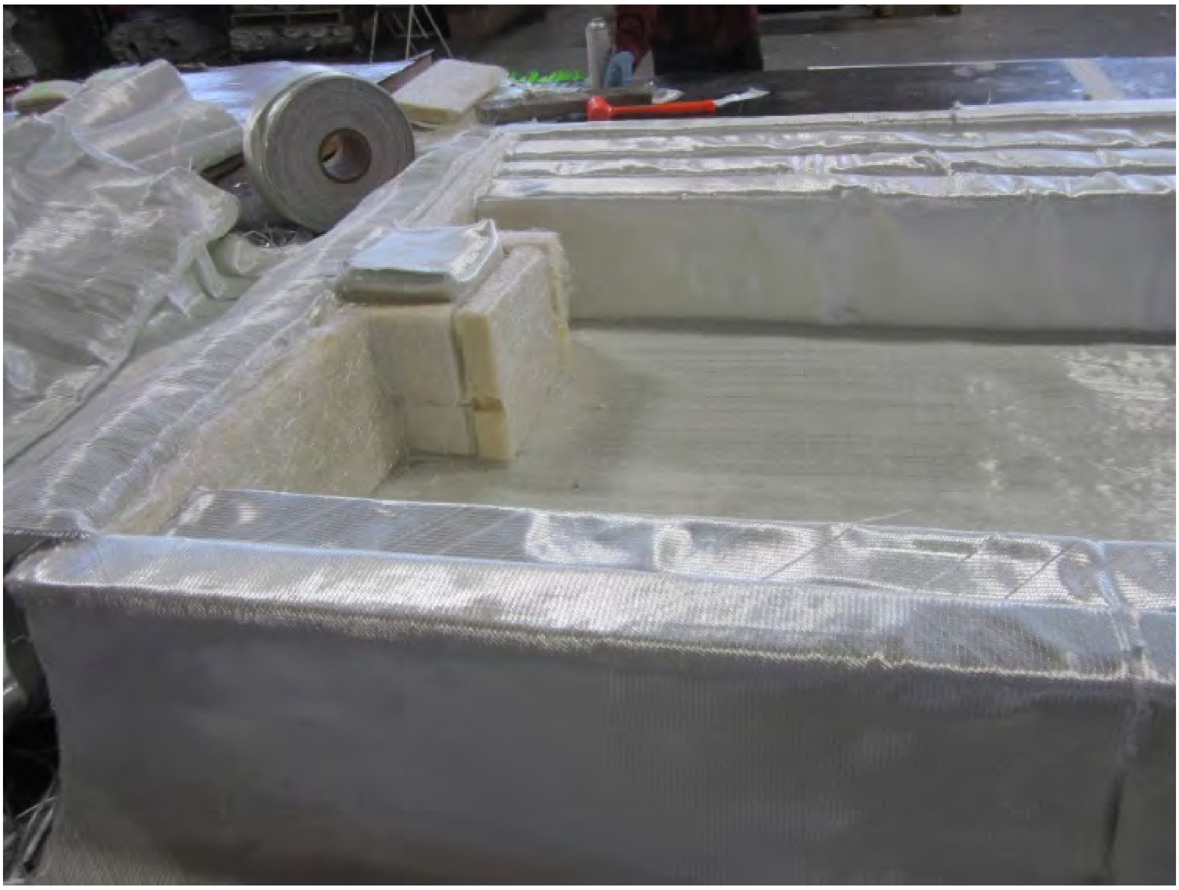

Figure 5: FRP Wicket Gate Fabrication Preparation

Once cured, the gate was removed from the mold and tested in a four point flexure test to validate the gate’s flexural capacity; see Figure 5. Though not tested to ultimate failure, the gate was tested to a capacity exceeding the allowable design value and was therefore approved for installation.

Figure 6: Flexure Test of Prototype FRP Wicket Gate

After the proof testing, the team met and discussed changes to improve the design. There were concerns about abrasion resistance on the upstream surface and side walls. To address this, ultra-high molecular weight polyethylene (UHMWPE) was incorporated as a wearing surface on the upstream face and sides of the gate. Another change was to make the entire gate high-visibility yellow. Two additional FRP composite wicket gates were manufactured to incorporate these changes. The first prototype was painted with a yellow coating to match the other gates.

The composite prototypes were manufactured at two thirds the cost of the timber gates with an estimated life of 50 years as opposed to the 10 to 20 years of the timber wicket gates. The gates were placed into service at Peoria Lock and Dam on the Illinois River in August of 2015, as shown in Figure 7, and have shown no signs of damage or deterioration during periodic inspection. The wicket gate marked the first major implementation of FRP composite structures into USACE navigation systems. The demonstration of significant cost savings in production and maintenance using the VARTM process created interest in further developing FRP composite materials for larger gate structures.

Figure 7: Installation of FRP Wicket Gate with UHMWPE

3. INTERNATIONAL COLLABORATION

3.1 PIANC Working Group 191

In 2015, PIANC established a working group, WG 191 – "Composites for Hydraulic Structures". Representatives from both WVU and USACE are members of this working group. This international group is tasked with identifying where composite materials provide a benefit over conventional materials for hydraulic inland navigation structures and to develop a report identifying best practices of how to use composite materials, summarizing case studies with pros and cons, and to compile guidance documents to aid engineers when using composite materials in the demanding environments of hydraulic structures. This working group has allowed researchers to collaborate and exchange information with other agencies.

3.2 USACE – Rijkswaterstaat MOA

A Memorandum of Agreement (MOA) between the Ministry of Transport, Public Works and Water Management of the Netherlands Rijkswaterstaat (RWS), and USACE was established in 2004. The purpose is to promote a long-term relationship between RWS and USACE on collaborative efforts of mutual benefit to the Netherlands and the United States. One of the stated goals of this MOA is to share best practices, lessons learned and expertise. Under this MOA there is a working group on Navigation Infrastructure. This working group meets monthly by phone and twice a year face-to-face. Navigation infrastructure topics include standardization, structural health monitoring, structural life extending practices, innovative materials, life cycle management, and sustainability of hydraulic structures.

3.3 Site Visit

During a meeting of WG191 in June 2017, RWS hosted a site visit to the world's largest FRP composite miter gates at Lock III in Tilburg the Netherlands on the Wihelmina Canal, Figure 8. Designed to resist a hydraulic head of 7.8 meters, each gate leaf on the downstream side is 6.3 x 12.3 x 0.51 meters. The design life for these miter gates is 100 years. The gates were fabricated using a VARTM process as monolithic units. The gates were manufactured similarly to the wicket gates with faceskins connected with shear webs separated by non-structural polyurethane foam. In this case, however, the fiber in the skins and the webs was continuous. The gate was fabricated with E-glass fabric and polyester resin for durability and cost. Design guidance followed for this effort included the Dutch Design Recommendation CUR 96.2003 and revision 2014, VORSTENBOSCH KRABBE, J.P. (2015).

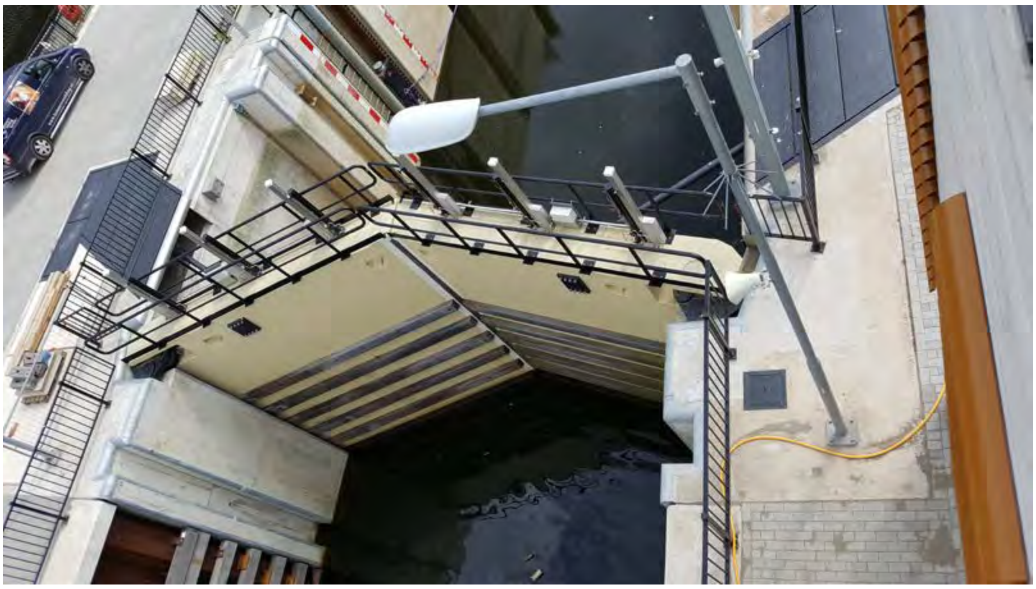

Figure 8: World's Largest FRP Composite Miter Gates at Lock III in Tilburg the Netherlands.

4. NEXT STEPS

Through lessons learned from previous FRP projects and technology exchange meetings, USACE intends to develop and install FRP gates and valves for inland navigation structures. USACE has a sizeable inventory of miter, tainter (radial), and vertical lift. These gates are larger and more complex in design than wicket gates. Each of these gates presents a unique challenge. While composite materials can provide solutions to many of these challenges, further development will require an incremental process to overcome the various challenges present in scaling up the design. Using information gathered from previous collaborations and implementing intermediate water control structures, many of these uncertainties can be addressed.

Discussions are underway with Rock Island District and the Illinois Waterway (IWW) to design, fabricate, and install an FRP composite culvert valve with UHMWPE slides as a follow up to the wicket gate. They have a current effort to redesign the existing steel culvert valves and incorporate UHMWPE slides. The current culvert valves (also known as slide gates) are a common design across multiple locks along the IWW allowing the valves to be used interchangeably. The current steel valves are designed for a maximum hydrostatic head of 39 feet and 12 feet wide by 9 feet tall. To open and close, the gates are operated with a hydraulic actuator attached to a strong back. Steel rollers provide the sliding mechanism. However, due to corrosion, these steel rollers are prone to seizing up. Once seized, the rollers wear against the slots resulting in flat spots on the rollers and damage to the slots. The new steel culvert valves replaced the rollers with UHWMPE slides to eliminate this issue. The plan is to design and fabricate an FRP composite culvert valve with UHMWPE slides for installation along with the new steel valves.

By designing a slightly larger gate than the composite wicket gates, a few challenges will have to be addressed. To meet flexural stiffness requirements, an embedded steel frame may need to be incorporated into the design of the culvert valve requiring an effective design methodology for hybrid steel structures to be established. To connect the actuator to the top of the gate and mechanically fasten the UHMWPE slides to the gate, additional embedded steel components may be needed. Methods for integrating these components into the composite gate while protecting them from corrosion from moisture diffusion will also need to be investigated. To determine the fatigue resistance of the gate, the fatigue resistance of the embedded steel frame, the FRP composite materials, and the bond between them will need to be estimated independently. Once these resistances are more defined, the fatigue resistance of the gate as a unit will be understood.

By following similar design procedures to the Tilburg miter gates and knowledge gained through the culvert valve, USACE researchers will be able to move to even larger gates with more complex designs. Due to their size and intended use, they will be subjected to greater loading conditions and stresses. The large size of these of gates will likely require them to be manufactured in large monolithic pieces using a VARTM process. Research and development will need to be conducted to limit the amount of pieces required to construct an entire FRP composite gate.

5. POTENTIAL RESEARCH TOPICS

5.1 Structural Health Monitoring

Structural health monitoring of inland navigation structures using strain gages and accelerometers has proven to be a valuable Operations and Maintenance (O&M) tool to monitor the functionality and integrity of these large structures. The VARTM manufacturing process allows for a new and innovative way to include sensors within the structure rather than adhered to it. Further research and development will be required in order to select the appropriate type of sensors, optimal sensor placement locations, and data acquisition techniques.

5.2 Inspection Techniques

When FRP composite materials are damaged, it may be difficult if not impossible to visualize the damage from the surface. Forms of damage can range from fiber failure, delamination between lamina layers, and expansion of voids initially caused by poor resin penetration. Non-destructive testing (NDT) techniques such as a digital tap hammer, ultrasonics, thermography, radiography, and shearography need to be explored in an attempt to not only discover possible damage areas such as delamination and voids, but to also quantify the damage.

5.3 Repair Techniques

If a damage area can be quantified as critical, then a repair of the structure will be necessary. Proper surface treatment is essential for a successful composite repair. Material removal can be accomplished by mechanical milling or with pulsed lasers. Depending on the nature of the composite, a patch, scarf patch, or step repair will need to be chosen as a repair technique. Once the damaged area has been excavated, the exposed surfaces will need to be further cleaned for the final repair patches. This is typically accomplished by plasma burning surface contaminates, using lasers to remove matrix material in order to expose fibers, or improving surface wettability for adhesives by photochemical reactions induced by UV-laser light. Once the surface is prepared, a patch is typically applied under vacuum and high temperature. Field repairs are typically done using a hot bonder whereas more complex repairs requiring a higher quality repair are done using an autoclave. Further research is necessary to develop what repair methods, materials, equipment, and cure time will be needed for particular structural components. Additionally, NDT and monitoring procedures will need to be developed to ensure the quality and integrity of the repair.

6. DESIGN GUIDANCE AND CONSIDERATIONS

The design guidance available for hydraulic steel structures, USACE (2014), was consulted as a template to follow for developing guidance for the design of hydraulic composite structures. Categories that will be incorporated from this guidance will include materials, member types, analysis methods, inspection methods, plans and specification, fabrication and erection procedures, design for fatigue and fracture, and design and detailing of connections. The guidance was also consulted to identify considerations unique to miter, tainter (radial), and vertical lift gates, USACE (2014).

6.1 Miter Gates

Miter gates are large vertical lock gates operating under moderate to high lift. Components of a miter gate include skin plates, girders, diagonals, quoin blocks, and gudgeon connections. During normal operations, the gates are exposed to hydrostatic loads, hydrodynamic loads, machinery loads, and barge impacts. When opening and closing, the gates are also exposed to significant torsional loading. Due to high hydraulic heads and constraints on maximum gate thickness from existing lock geometries, embedded steel frames may be need in to meet deflection requirements. Depending on the size of the gate, fabricating the gates as monolithic units may not possible which would require modular fabrication techniques instead. Guidance for designing connections between the composite gates to operational machinery and gudgeon assemblies for both strength and fatigue resistance will be needed.

6.2 Tainter Gates

Tainter gates used for spill way control on navigation structures are exposed to significant discharge. Components of a tainter gate include radial skin plates, horizontal girders, end frames, and trunnions. The gates experience loads from hydrostatic pressure, gate lifting systems, ice impacts, side seal friction, trunnion pin friction, hydrodynamic pressure, and wind. Due the discharge, the gates experience significant vibrational loading. While composite materials can be optimized to provide damping against such vibrations, guidance must be provided to design for sustained fatigue to prevent delamination and mirco-cracking. Additionally the complex geometries of the gates require that they be fabricated from separate components. Special attention will be paid to designing the anchorage systems for attaching lifting cables and actuators to the profile of the gates.

6.3 Vertical Lift Gates

Vertical lift gates are used as both lock and valve gates for low to moderate heads and can be of an overhead or submersible configuration. The lift gates components include skin plates, framing systems (girders, trusses, tied arches, or vertical framing), and end supports that allow vertical motion (fixed-wheels, rollers, or slide). Vertical lift gates experience many of the same loads as miter gates but are also subjected to vertical loads from the lifting machinery, thermal differentials, and wind loads in the case of overhead gates. Embedded steel plates or frames may be necessary to prevent pull out of the lifting connection.

7. CONCLUSIONS

Successful demonstrations utilizing FRP composite materials for USACE navigation infrastructure components has shown the viability and cost reductions that can be realized with these materials. Collaborations with other international waterway agencies has provided further evidence that FRP composite materials can be used to fabricate even larger navigation structures. By leveraging the lessons learned in these successes and collaborations, as well as current industry innovations, a path towards the design and procurement of even larger FRP composite structures has been developed. As part of this path forward, knowledge gaps related to structural health monitoring, inspection, and repair techniques will be filled in. Ultimately, the knowledge obtained from this effort will be developed into comprehensive design guidance to equip engineers with the tools needed to widely implement FRP composite navigation structures with confidence.

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NUMERICAL INVESTIGATION OF THE IMPACT OF THE INLAND TRANSPORT ON BED EROSION AND TRANSPORT OF SUSPENDED SEDIMENT : PROPULSIVE SYSTEM AND CONFINEMENT EFFECT

by

Sami Kaidi[1, 2], Hassan Smaoui[1, 2], Philippe sergent[1], Fabrice Daly[1]

ABSTRACT

In the last few years, transport by inland waterways has experienced rapid growth thanks to the development of a new generation of larger and more powerful ships. This new generation of ships has a high ecological quality in terms of CO2 emission. However, their impact on the navigation environment has increased especially on the erosion of channel bed and banks. This phenomenon of erosion is mainly caused by the turbulent flow around the ship caused by its movement as well as its propulsive system. Hence, for a better prediction of this phenomena, it is essential to simulate accurately the flow around the ship.

The evaluation of the impact of the ship passage on the erosion phenomenon has been treated in the past by different methods: empirical, analytical and numerical. In the present work we propose to study this phenomenon of erosion applied on the channel bed numerically by the method Computational Fluid Dynamics (CFD). Several values of depth (h) to draught (T) ratio, ship advance ratio and sediment size (d_{50}) were simulated to assess the impact of each parameter on the bed shear stress. The results of this work clearly show the influence of each parameter tested.

1. INTRODUCTION

Transport by inland waterways is considered as a complement to rail and road transport. In the last year's a lot of attention. This mode of transport is known by its ecological quality in term of CO2 emission compared to road and rail transport. However, despite the reduced amount of CO2 emitted, this mode of transport still has a significant negative impact on the environment, which is the erosion of banks and the bed of inland waterways as well as the re-suspension of existing polluting particles and contaminants. This impact becomes more and more visible with the arrival of the new generation of ships with large size and their very powerful propulsive system.

The presence of suspended sediment in inland waterways leads to difficult problems for the development and maintenance of channels. To these problems are added the quality of the water. In fact, sediments trap many elements such as metals of industrial origin. There can be transport or accumulation of these pollutants which may be re-suspended under the effect of hydrodynamics of the water often disturbed by ships passage. These re-suspension can contribute to transport of pollutants from a polluted area to a unpolluted area.

The understanding and control of interactions at the water-sediment interface are extremely complex due to the presence of several processes of natures and spatiotemporal scales very different. The hydro-sedimentary processes are governed by the action of friction exerted by the water on the bed of the channel. It is generally accepted that sediment transport is carried out in two modes: bedload on the channel bed and suspension in the water.

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Numerical modeling of the sediments suspension phenomena is often carried out on a large scale by models such as Saint-Venant or Boussinesq SMAOUI ET AL. (2011). The recourse to this type of model is mainly related to the simplifying assumptions adopted. These models use empirical formulas for estimating shear stress applied on sediments using average velocities. Recently, and thanks to the rapid development of computing resources, fully models based on Navier-Stokes equations are used to model sedimentary transport, BROVCHENKO ET AL. (2007). The coupling between the fluid model and the sediments transport models can be strong (simultaneous resolution of the both equations) or weak (alternative resolution). The advantage of these models is the high precision of the shear stresses estimation on the channel bed.

Hence, in the present work a fine numerical study was conducted to assesses the impact of the inland transport on the inland waterways. The influence of the under keel clearance, the propeller advance and size of sediments coefficient have been tested for several. In this study to simulate the flow around the propelled ship the (CFD) based on Unsteady Reynolds-Averaged Navier-Stokes (URANS) was used. This model was coupled with a sedimentary transport model to simulate the re-suspension of sediments. The both models were verified and validated using an experiment data. For this investigation the flow is considered with free surface and turbulent. An inland ship with twice propellers and four rudders was selected for this investigation.

2. **MATHEMATICAL MODELS**

2.1 Mathematical models

2.1 Fluid flow equations

The simulation of the flow around the ship was carried out using the steady Reynolds Averaged Navier-Stokes (RANS) equations for incompressible flow with free surface. The governing equations for mass and momentum conservation are given as follow:

$$\frac{\partial \rho}{\partial t} + \frac{\partial(\rho u_i)}{\partial x_i} = 0$$

$$\frac{\partial(\rho u_i)}{\partial t} + \frac{\partial(\rho u_i u_j)}{\partial x_j} = -\frac{\partial P}{\partial x_j} + \frac{\partial(\rho \overline{\acute{u}_i \acute{u}_j})}{\partial x_j} + \frac{\partial}{\partial x_j}\left[\mu\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} - \frac{2}{3}\delta_{ij}\frac{\partial u_i}{\partial x_i}\right)\right]$$

$$\rho = \sum_{n=1}^{2} \alpha_n \rho_n \ , \qquad \mu = \sum_{n=1}^{2} \alpha_n \mu_n$$

Where$x_i (i = 1,2,3)$ are Cartesian coordinates; ρ is the density of the water and t is the time; $u_i (i = 1,2,3)$ are Cartesian velocity components, P and μ are pressure and dynamic viscosity respectively; α is the phase fraction and $n = 1,2$ represents the fluid phase number (water and air); δ_{ij} is the Kronecker delta; $\acute{u}_{i(j)}$ denotes the fluctuating velocity; $-\rho\acute{u}_i\acute{u}_j$ is the average Reynolds stresses which can be written in the following form:

$$-\rho\overline{\acute{u}_i \acute{u}_j} = \mu\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right) - \frac{2}{3}\left(\rho k + \mu_{ij}\frac{\partial u_i}{\partial x_i}\right)\delta_{ij}$$

The (SST) $k - \omega$ turbulence model was chosen to close the Navier-Stokes equations and to predict accurately the hydrodynamic forces near–wall regions. The air-water interface in the free surface was modeled using the Volume of Fluid method (VOF).

2.2 Sediments transport equations

The governing equation for suspended load is a standard convection diffusion equation. The numerical treatment for this transport equation is similar to the momentum equations.

$$\frac{\partial \rho c}{\partial t} + \nabla . [\rho(u - u_s)c - \Gamma \nabla c] = 0$$

$$u_s = \langle 0, 0, -w_s \rangle$$

Where C is the volumetric sediment concentration, Γ is the difusivity of sediment and $\boldsymbol{w_s}$ is the settling velocity of sediment particle. For suspended sand particles and depending to the sediment diameter range, the following equations are used to compute $\boldsymbol{w_s}$, VAN RIJN, L. C. (1984):

$$\begin{cases} w_s = \dfrac{(s-1)gd_{50}^2}{18\nu} & if\ \ d_{50} < 100\ \mu m \\ w_s = 10\dfrac{\nu}{d_{50}}\left(\left[1 + \dfrac{0.01(s-1)g\rho d_{50}^2}{\nu^2}\right]^{0.5} - 1\right) & if\ 100\ \mu m\ <\ d_{50} < 1000\ \mu m \end{cases}$$

with $s = \rho_s / \rho$

The resolution of the transport equation assumes knowledge of the boundary conditions especially on the free surface and the channel bed.

In the present work, the absence of the transfert of the sedimentery material is assumed on the free surface and a constant concentration of sediment is assumed on the channel bed (C_a). Where:

$$C_a = \frac{0.035}{\alpha}\rho_s \frac{d_{50}}{(z_a - z_0)D_*^{0.3}} \frac{\left(\tau_b - \tau_{b,cr}\right)^{1.5}}{\tau_{b,cr}}$$

Where, $\boldsymbol{\alpha}$ is an empirical paramètre, $\boldsymbol{z_0}$ is the bed roughness, $\boldsymbol{\tau_b}$ is the shear bed stress generated by the flow near to the bed. $\boldsymbol{\tau_{b,cr}}$ the critical shear bed and $\boldsymbol{D_*}$ is a parameter depending to the sand particles given as:

$$D_* = d_{50}\left(\frac{(s-1)g}{\nu^2}\right)^{1/3}$$

The critical shear bed stress is deduced from the Shields curve. In the present investigation we use the formula proposed by YALIN, M. S. (1977

$$\tau_{b,cr} = (\rho_s - \rho)gd_{50}\left(\frac{0.186}{1 + 0.2\varpi} + 0.045\left(1 - 0.98e^{-0.01\varpi}\right)\right)$$

Where $\boldsymbol{\varpi}$ is a dimensionaless parameter :

$$\varpi = \left[\frac{(s-1)gd_{50}^3}{\nu^2}\right]^{0.5}$$

2.2.1 Sediments scaling method

This investigation was done at the scale 1/25. For the sediments scaling, the following formula determined from the equation of the Shields number was used:

$$n_s = \frac{n_L}{0.29} \approx 86.2$$

Where, $n_s$is the sediment scale factor and n_L is the length scale factor, here, equal to 25.

3. SHIP DESCRIPTION

The ship selected for this investigation is an inland container cargo of 135m of long and 11.40m of width (see Fig. 1). This ship is equipped by two propellers type VP1304 with five blades and four rudders type fishtail (also called Schilling rudders). The ship form was put on the scale 1/25th. Its characteristics dimensions are presented in Table 1.

Figure 1: Ship geometry

4. Table 1 Geometric parameters of ship hull

	Length (L_{PP})	Beam (B)	Draft (T)	Block coefficient (C_B)	Wetted surface (W_S)	Cross area of ship (C_S)
Real model	135 m	11.4 m	2.5 m	0.899	2104.8 m²	34.114 m²
Scaled model (1/25)	5.4 m	0.456 m	0.1 m	0.899	3.367 m²	0.0545 m²

Table 1: Geometric parameters of ship hull

4. VERIFICATION AND VALIDATION OF THE NUMERICAL MODELS

4.3 Verification and validation of the numerical models

The verification and validation of the hydrodynamic model of the hull and propellers were presented and detailed in previous works KAIDI ET AL. (2017) and KAIDI ET AL. (2018). Hence, for this investigation only the verification and validation of the sediments transport model and the scaling method were considered. For that, the experimental data of Vin Rign trench tests were used. Fig. 2 shows the configuration setup and its dimensions.

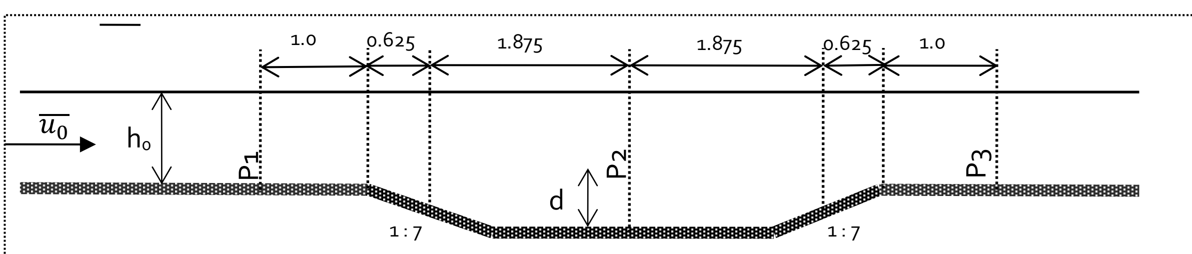

Figure 2: trench setup and dimensions

The velocity inlet condition was applied on the trench inlet, while the pressure outlet condition was applied on the outlet boundary. The trench bottom was considered as roughness wall (Ks=0.025). The sides and the top of the trench were modeled as a symmetry plane condition.

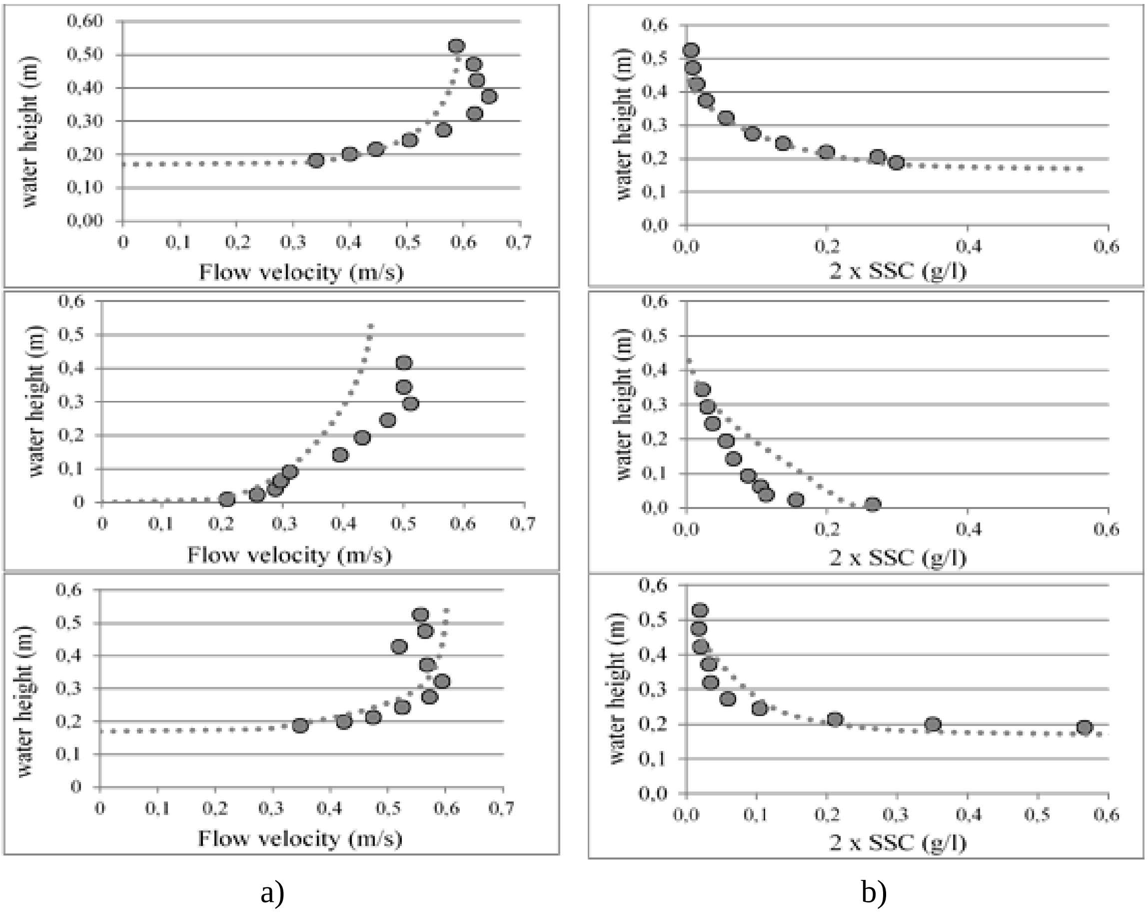

Figure 3: Comparison between numerical and experimental results of the physical model : a) flow velocity and b) the SSC

The vertical variation of the flow velocity and the suspended sediment concentration (SSC) computed in three different locations of the trench (in the inlet, center and outlet of the trench) were compared to the measured data (see Fig. 3). A good agreement between the computed results and data was observed in the inlet and outlet of the trench. Where the relative error for the flow velocity don't exceed 8% observed near to the free surface. For the SSC the relative error is about 5%, observed at the middle of the water column. However, in the trench center a discrepancy between the computed and measured data was noted in the flow velocity and SSC. This discrepancy is probably due to the ignorance of the free surface and can depends strongly to the turbulence model used. The same remarks and observations were noted for the scaled model 1/25 (see Fig. 4.).

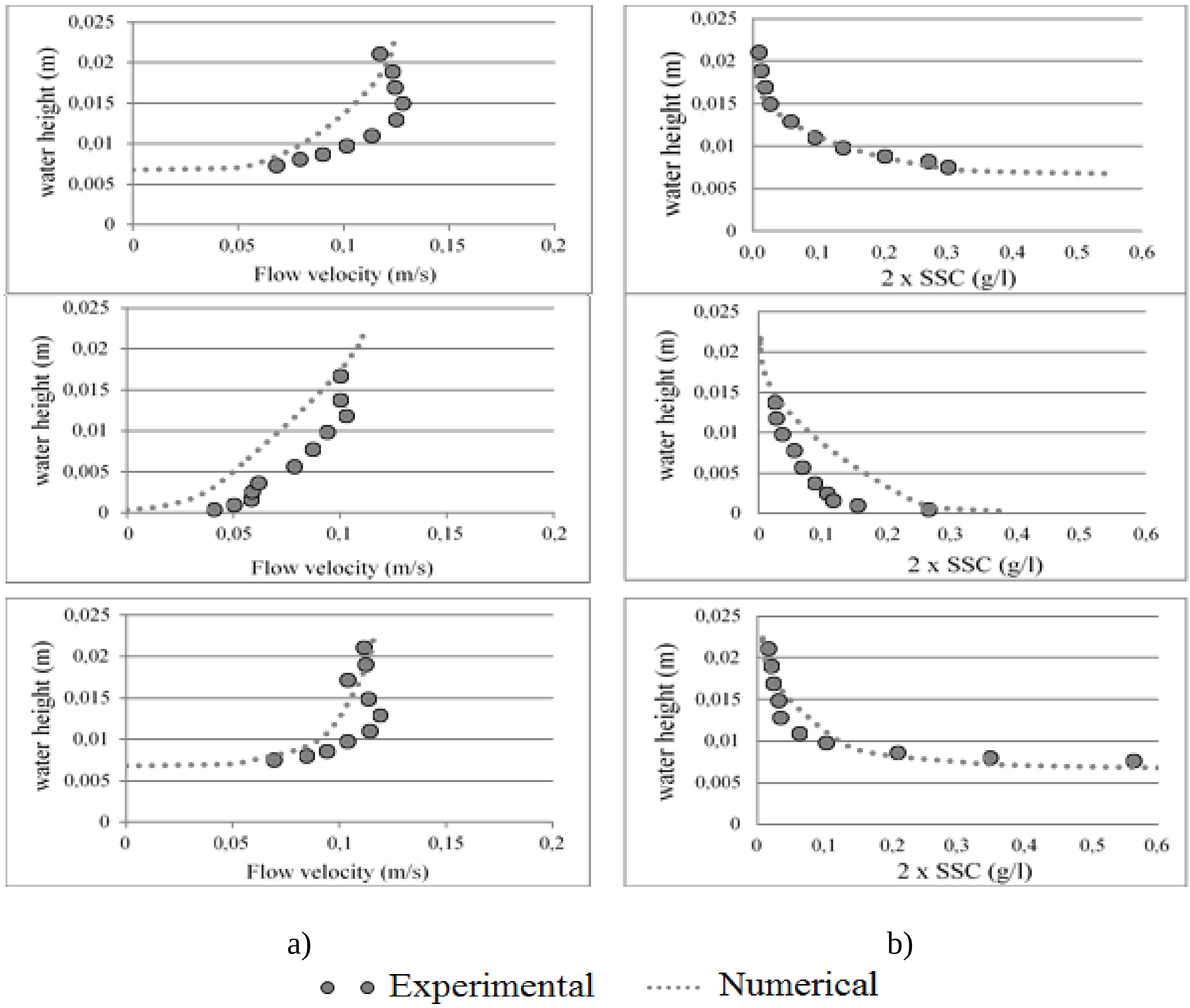

Figure 4: Comparison between numerical and experimental results of the scaled model : a) flow velocity and b) the SSC

4.2 Tested configurations and corresponding boundary conditions

In the present study, all tested configurations are summarized in the table 2. Three parameters were tested : the depth to draught ratio (h/T), the ship advance ratio and the sediments size.

<table>
<tr><th></th><th>h/T=1.2</th><th>h/T=1.5</th><th>h/T=2.0</th><th>h/T=3.0</th></tr>
<tr><td>J=0.7</td><td>0.55 (m/s)</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td rowspan="2">J=0.9</td><td>0.55 (m/s)</td><td rowspan="2">0.55 (m/s)</td><td rowspan="2">0.55 (m/s)</td><td rowspan="2">0.55 (m/s)</td></tr>
<tr><td>1 / 1.6 / 2.4 / 3 (10^{-6}m)</td></tr>
<tr><td>J=1.1</td><td>0.55 (m/s)</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td>J=1.3</td><td>0.55 (m/s)</td><td>X</td><td>X</td><td>X</td></tr>
</table>

Table 2: Tested configurations : The ship speed Vs (m/s), the depth to draught ratio (h/T), the advance ratio (J) and the sediments size (m)

5. RESULTS

5.1 Effect of operating propellers on the sediments suspension

5.1.1 Comparison between bed shear stress caused by a towing and a propelled ship

In this section the impact of the propulsive system on the shear bed stress was studied. To carried out this study, a comparison between a loaded bed during the passage of a towed ship and propelled ship was presented in two different water depth. Fig. 5 (a) shows the shear bed stress generated by the passage of a towed (lift) and propelled (right) ship for h/T ratio of 1.2, while the Fig. 5 (b) shows the shear bed stress generated by the passage of a towed (lift) and propelled (right) ship for h/T ratio of 1.5. The ship speed was set for the both tests to 0.55m/s and the advance ratio was set to 0.90 (470 RPM).

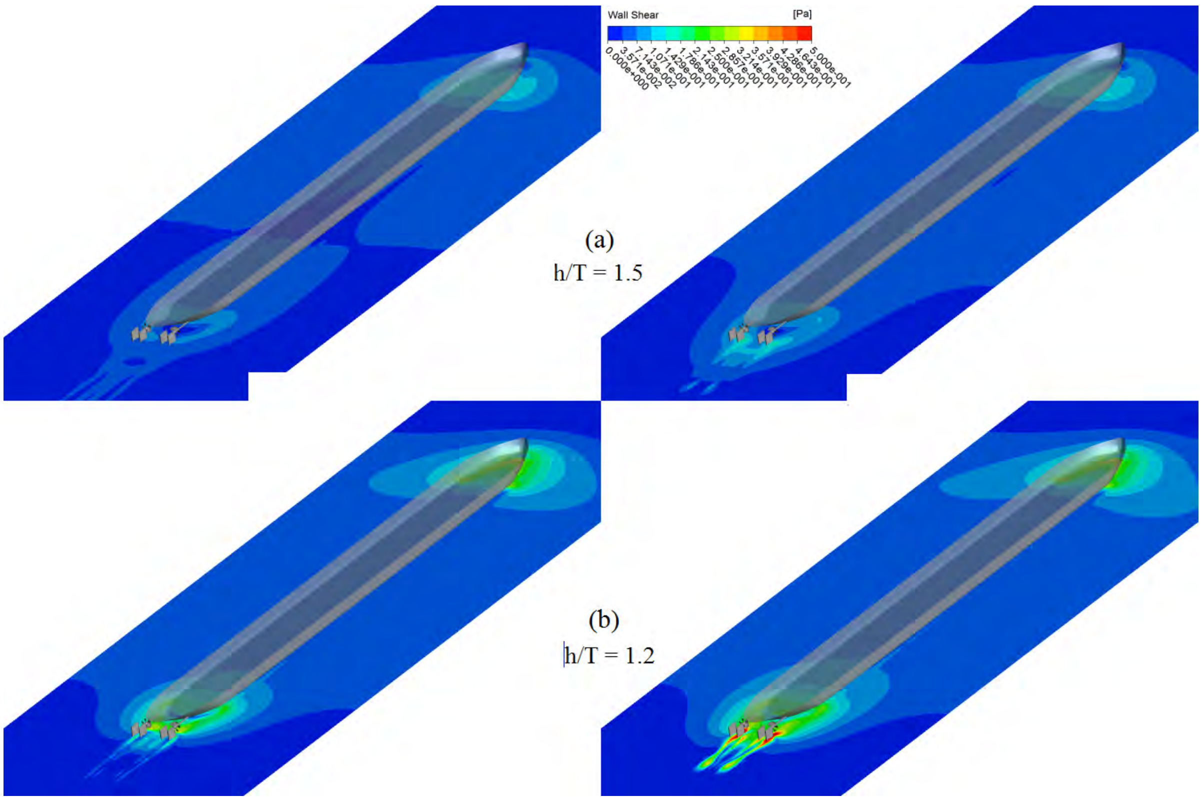

Figure 5: Bed shear stress for towed and propelled ship : a) h/T =1.5 and b) h/T=1.2

It can be seen clearly from these figures that the bed shear stress observed in the ship bow for the both h/T ratio is almost unchanged. While, the bed shear stress in the ship stern zone increases especially around the propellers. The Fig. 6 illustrates a comparison between the computed SSC along the midline of the channel for a towed and propelled ship in very shallow water case. The SSC was taken at the height of 0.015m from the channel bed. The both curves have the same allure in the ship bow, however, in the ship stern the SSC increases significantly to reach a 1.52 g/l which corresponds to 3.6 times the SSC generated by a towed ship. These curves also give an idea about the repartition of the SSC and the length of the impacted area behind the ship.

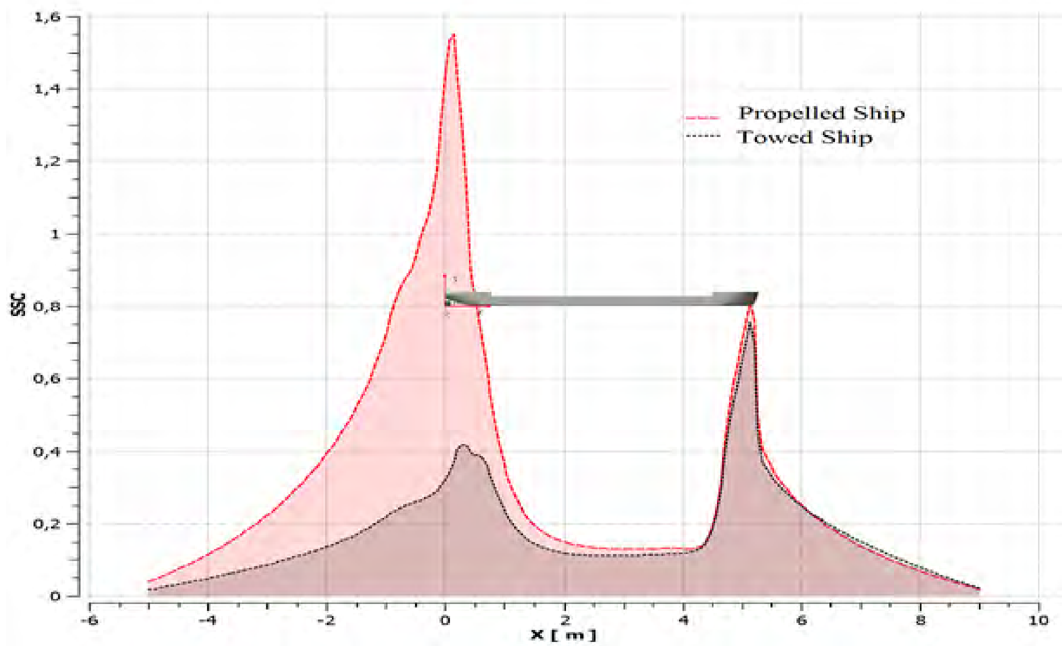

Figure 6: SSC (g/l) distribution on the midline of the ship for towed and propelled ship

5.1.2 Effect of the ship advance ratio (propeller thrust)

Several advance ratios (0.7, 0.9, 1.1 and 1.3) were tested to assess the influence of this parameter on the bed shear stress and the suspension of the sediment. The ship speed is supposed constant and equal to 0.55m. Hence, the variation of the advance ratio is done by the variation of the propeller turning rate. This investigation was done in very shallow water h/T=1.2.

For the same ship speed it can be observed that when the advance ratio is varied the concentration of the suspended sediment vary in the bow and the stern of the ship. Fig.7 presents the SSC variation along the midline of the channel. It can be seen that the SSC increases with advance ratio decrease. The increase noted in the bow zone is weak compared to the ship stern zone, where, the SSC computed for the lower value of the advance ratio is greater than 5 times the SSC computed for the higher value. This increase is caused essentially by the propeller jet which increase by propeller turning rate increase.

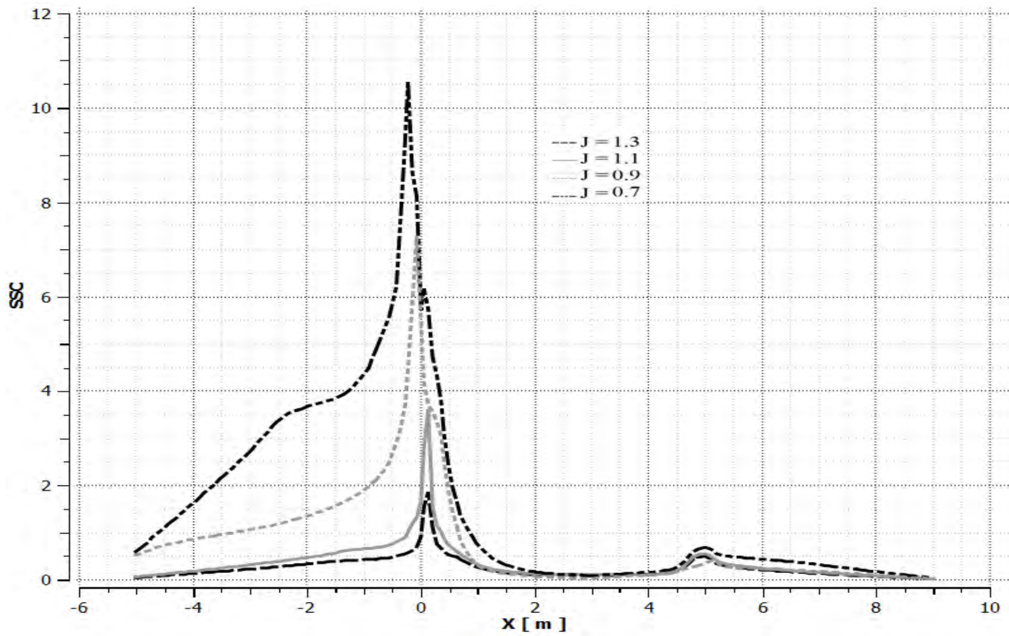

Figure 7: SSC (g/l) distribution on the midline of the left propeller according to advance ratio

5.2 Confinement effect

5.2.1 Shear bed and SSC distribution as a function of h/T ratio

The shear bed stress distribution was plotted for four h/T ratios. Here, the ship speed was set to 0.55m/s and the propeller turning rate was set to 470 tr/min (*J*=0.90). The ship draught was considered unchanged and was set to 0.1m. Hence, to test the confinement effect only the water depth was changed. The tested values of h/T ratio are 1.2, 1.5, 2.0 and 3.0 where, each value represent a type of waterways (confined, shallow, medium deep and deep waters). These values of h/T ratio correspond to the following depth Froude numbers (0.101, 0.090, 0.078 and 0.064).

The Fig.8 below show the bed shear stress caused by the passage of propelled ship and its distribution according to h/T ratio. From these figures, it can be observed that the propeller effect is felled by the bed only in very shallow, shallow and medium deep waters. However, this effect is more visible when the ratio h/T is less or equal to 1.5 (shallow water). This effect is more important when the ship is sailing in very shallow water (h/T =1.2), where, the computed shear stress value is 3.5 times more important compared to the shallow water and 12 times compared to medium deep water. In deep water the impact of the ship passage and its propulsive system is insignificant and the numerical results show that the hull effect is dominant than the propeller effect.

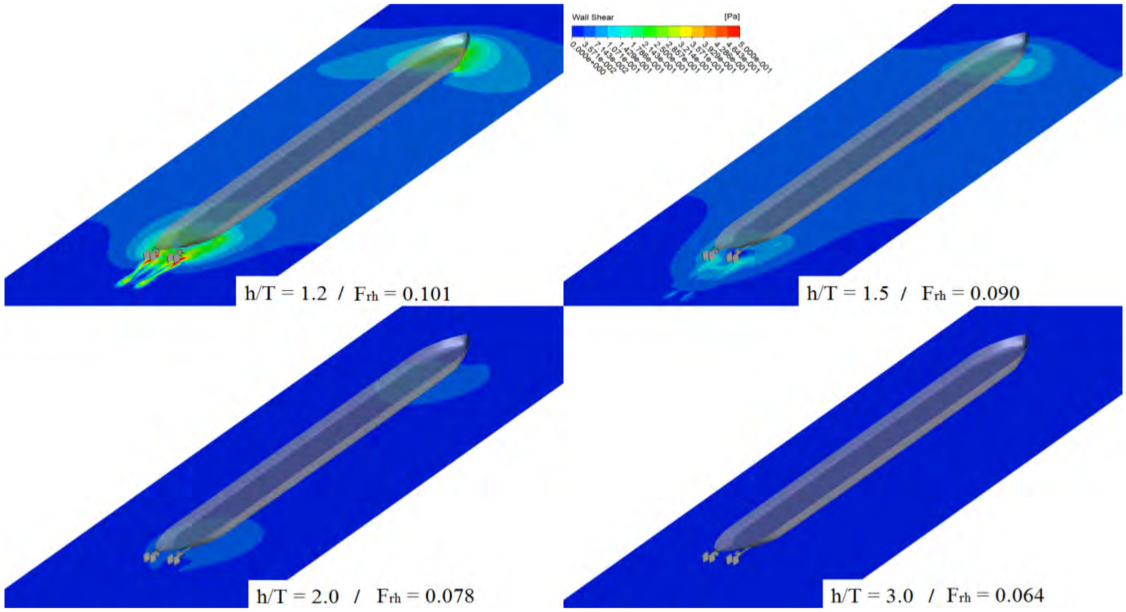

Figure 8: Bed shear stress according to the h/T ratio

The Fig. 9 shows the concentration of the suspended sediment along the midline of the ship located at a height of 0.015m from the channel bed. From these curves it can be noted that the SSC is very important in the both zones where the bed shear stress is significant. From these results we note that in all cases except in very shallow water, the concentration in the bow and the stern of the ship is almost the same. In very shallow water the SSC is considerable in the ship stern than in the bow, where, the computed SSC on the ship stern is almost doubled regarding to the ship bow. The main raison of this amplification is the accelerated propellers jet that erode the channel bed. Compared to the shallow water, the SSC computed in the very shallow water is 7 times larger. In medium and deep waters the SSC is about 0.22 and 0.10 g/l respectively.

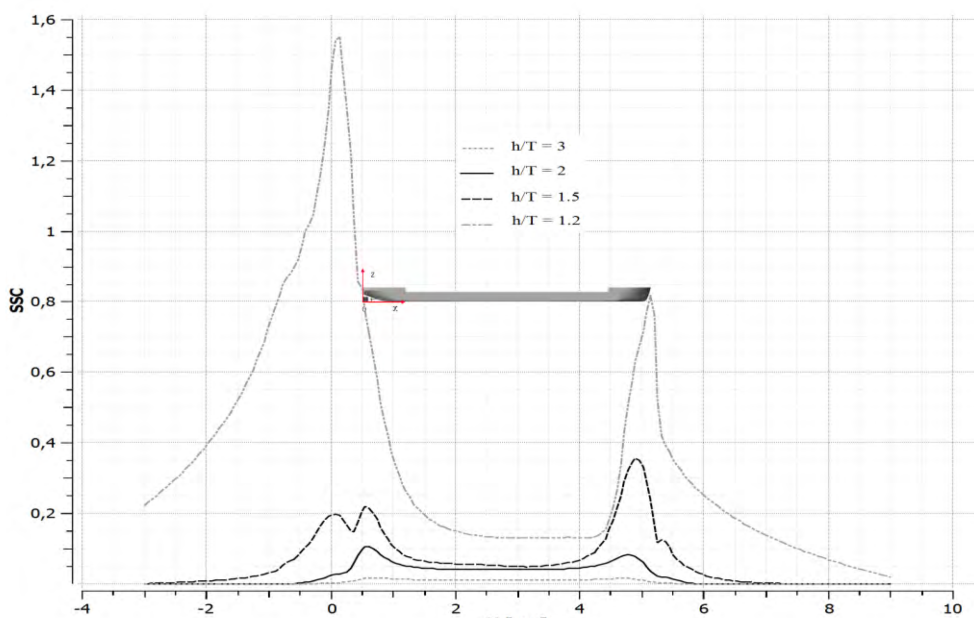

Figure 9: SSC (g/l) distribution on the midline of the ship according to the h/T ratio

5.3 Sediment size effect

The sediment size is one of the most parameters that can be taken into account to assess the SSC evolution and distribution. In this section we test four scaled sizes of sediments, $1.0*10^{-6,}$ $1.6*10^{-6}$, $2.4*10^{-6}$ and $3\ .0*10^{-6}$m (that corresponds to $0.0864*10^{-3}$, $0.138*10^{-3}$, $0.207*10^{-3}$ and $0.260*10^{-3}$ m). The ship speed, the ship advance ratio and the h/T ratio were set to 0.55m/s, 0.90 and 1.2 respectively.

The Fig. 10 shows the SSC distribution caused by the ship passage on the fluid domain. From this Figure it is observed that when the sediment size is small, the impacted zones are more larger especially at the ship stern. The numerical outcomes of the SSC are plotted along the midline of the ship and presented in the Fig. 11. It is shown that the SSC increase with sediments size decrease to reach 2 g/l for the smaller size of the sediment, while, the value of the SSC is less than 0.8 g/l in the case of larger size.

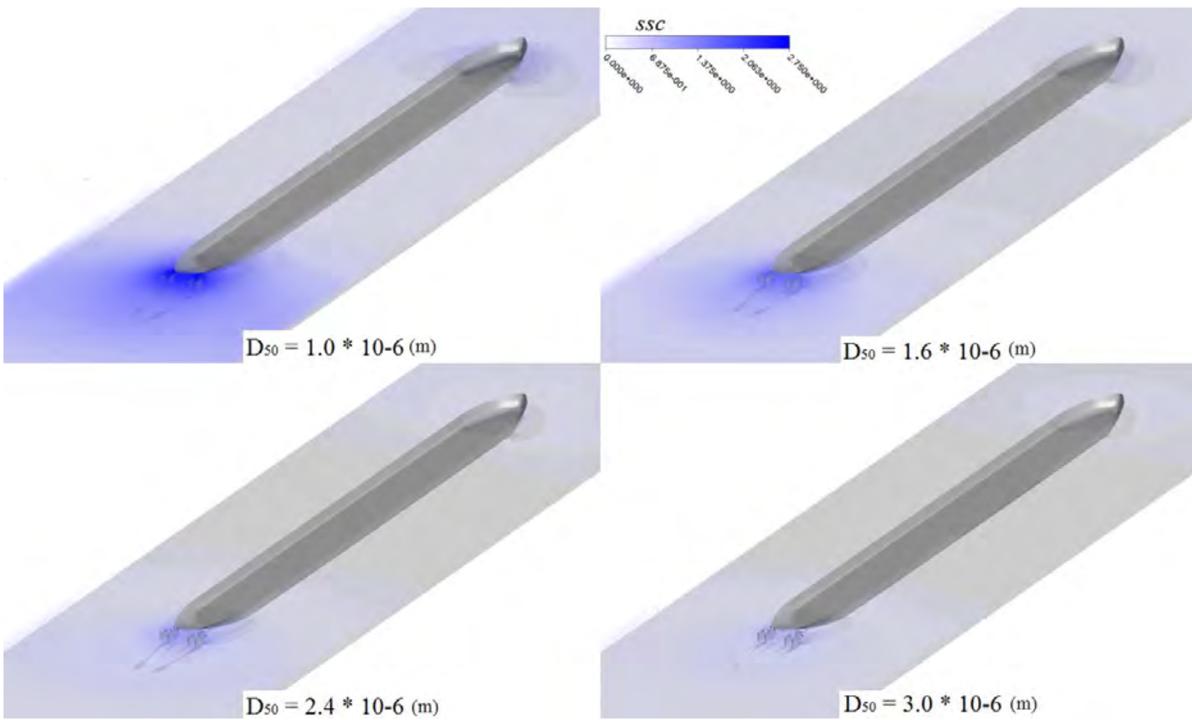

Figure 10: Volume representation of the SSC (g/l) distribution on the water according to sediments size

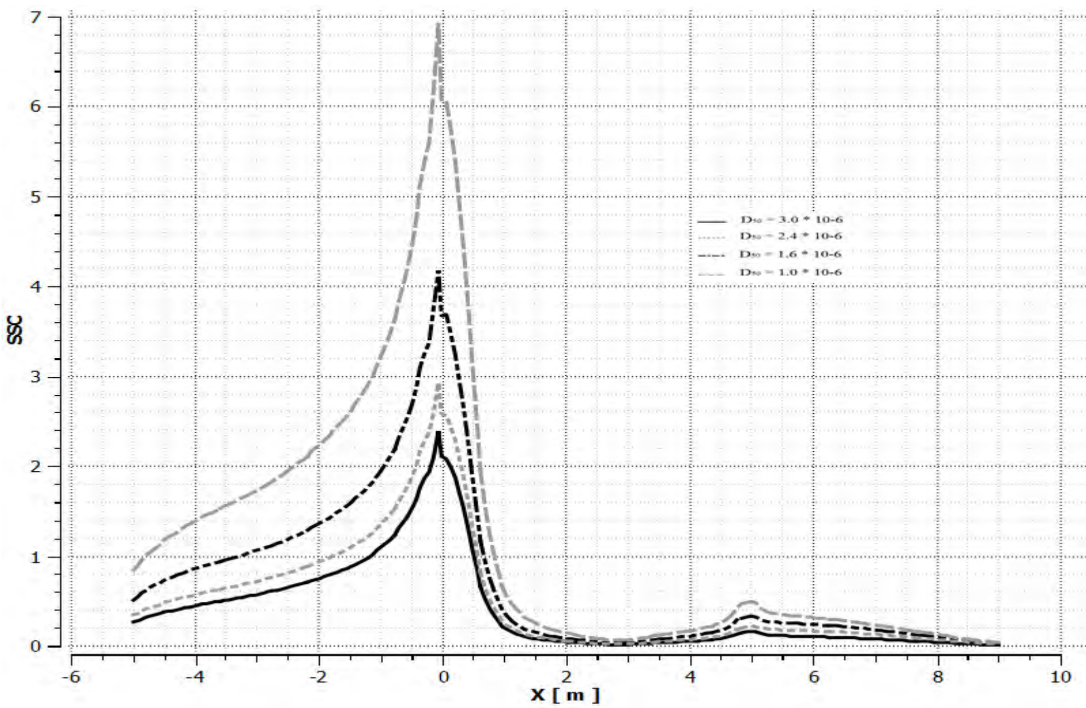

Figure 11: SSC (g/l) distribution on the midline of the ship versus sediment size

6. CONCLUSION

In this paper the evaluation of the impact of the inland traffic on the waterways was treated. The effect of the advance ratio of the ship, the h/T ratio as well as the sediment size was studied using a unsteady CFD model based on Navier-stokes equations with free surface. The (SST) $k - \omega$ turbulence model was used to estimate the viscous forces especially on the channel bed. The frame motion technical was selected to simulate the propellers turning. In this work, the mesh quality around the ship (hull, propellers and rudders) was chosen basing on previous work. The mesh quality on the channel bed was chosen basing on y+ value. The optimal mesh was used to validate the hydrodynamic model and the comparison with measured data showed a good agreement between the both results.

From this investigation it was presented the ability of the coupled model CFD – Sediment transport to predict accurately the suspended sediment concentration induced by the inland navigation in full and scaled models. It can concluded from the numerical outcomes of this work that all tested parameters depends on each other. Where, the propulsive system has an significant impact on the channel bed erosion only in shallow and very shallow water. This impact is visible principally in the ship stern zone. In medium deep water and deep water the effect of the propulsive system is negligible and only the hull effect is observed. From this investigation it was also noted that. It also was shown during this study that the channel bed erosion increase considerably when the advance ratio and the sediment size decrease.

Observations and remarks noted through this preliminary investigation can help to understand the different physical phenomenon related to the inland navigation and can also help to improve the mathematical models for channels design and channel dredging.

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NUMERICAL MODELING OF BANK EROSION DUE TO FLUVIAL TRAFFIC

by

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ABSTRACT

In this paper, we have developed a numerical model to estimate lateral bank erosion due to fluvial traffic. This estimation takes into account the river flow, the return current and the wake waves generated by navigation. The shear stresses due to river flow were calculated from Delft3D numerical code, while the shear stress due to navigation was calculated explicitly according to the characteristics of the wake waves (angle, period, amplitude). The algorithm allows coupling between these two shear stresses was inspired from methods usually used for the wave-current interaction implemented in marine environment. The model developed was applied to the real case of the river Oise (south of the city of Compiègne, France). The analysis of simulation results showed that the part of erosion due to river traffic is much smaller than that due to river flow. On the other hand, this paper concludes that the intensification of river traffic plays an important role in river bank failure. This problem will be inverted soon by the authors of this paper.

1. INTRODUCTION

Inland waterway transport fully meets the objectives of sustainable development since it allows pollutant emission reductions compared to road transport. According to VNF (Voies Navigables de France), the 2009 regional river traffic prevented the emission of 37,000 tons of CO_2 and consumed only 14.3 million liters of fuel, namely a consumption divided by 2.4 compared to the road transport. However, the development of this mode of transport will certainly have economic and ecological consequences for rivers that will be frequently used by river convoys. In fact, the intensification of fluvial traffic will generate wake waves (which will depend on the speed, size and structure of the boats and the size of the channels, etc.) responsible of bottom banks erosion. The consequences of this erosion on the aquatic environment are numerous: destruction of habitats, increase in turbidity of water and sedimentation, release of nutrients (phosphorus and nitrogen) that promote the proliferation of algae. The integrity of the land and the value of the riparian properties can also be negatively affected by erosion. To these environmental impacts will be added the consequent costs of the frequent maintenance of the network by dredging operations to maintain the depths of the navigation channels and to ensure flow conditions in times of flood or restorations for the Banks.

In this context, the CEREMA and the UTC (Université de Technology de Compiègne) conducted in 2006 (PHAM VAN BANG et al., 2007) a measurement campaign on two sites (Canal de la Sensé and the Seine river) to study the impact of boat passage on sediment transport and shoreline erosion. It is obvious that measurements alone are not sufficient to quantify all flow variables due to lack of information. For example, it is very difficult, if not impossible, to measure the *in-situ* turbulent fluctuations of a variable because of the wide variety of spatio-temporal scales contained in the turbulence phenomenon. These gaps can be supplemented by results from numerical models that allow the quantification of almost all the variables of the flow (intensity of the turbulence, shear

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stress, speed of friction ...). In this context, a simple 1D vertical numerical model (1DV) was developed and applied to the measurements zones (SMAOUI et al., 2011). This model was able to reproduce some features of the shear flow by boat speed at the surface and provide information on other parameters difficult to measure *in-situ*, but they could not answer all flow questions and bankserosion generated by the wake waves due to fluvial navigation. Therefore, it is necessary to consider the development of at least 2D for a more or less realistic study of the flows (fluvial flow intensity, bank characteristics, water height ...).

To complete the numerical study, in this paper, we propose a 2D numerical model to estimate shoreline erosion induced by combination of stream flow and a flow generated by the passage of boats (current return and waves of wave action). Indeed, the banks are eroded naturally by the shear stress due to fluvial flow. In this work, we integrate the shear stresses due to the presence of boats in navigation: namely the shear stress due to the return current and the shear stress due to the wave waves induced by the boats. The amount eroded is proportional to the maximum shear rate at the bottom (τ_{max}). It is clear that a good estimate of this quantity passes through a good parameterization of τ_{max}. On this part of the work, we were inspired by the work on the wave-current interaction in the marine environment (MOUAKKIR, L et al. 2010, H.; SMAOUI, H. & KAIDI, S. 2017).

2. **NUMERICAL MODELING**

2.1 Total shear stress

The displacement of a boat on the water surface creates the waves and the return velocity. These two hydrodynamic parameters interact in turn with the current of the stream. In fact, it is the resultant of these three parameters which affects the bank thus causing its destruction. To estimate the length of lateral erosion of the bank, it is therefore necessary to consider the resulting shear stress instead of the shear stress due to the stream flow only.

To take into account the effects of the waves, the problem is thus reduced to calculate the modulus of the resulting shear stress denoted by $\vec{\tau}_t$. This constraint is the vectorial sum of the constraint due to the current (denoted $\vec{\tau}_c$) and that due to the wake wave noted $\vec{\tau}_w$. To do this, we drew on work on the interaction of wave-currents in the marine environment (MOUAKKIR, L. et al., 2010). Figure (1) schematizes the operating principle of the wave-current interaction where the waves propagate with an angle φ with respect to the current direction.

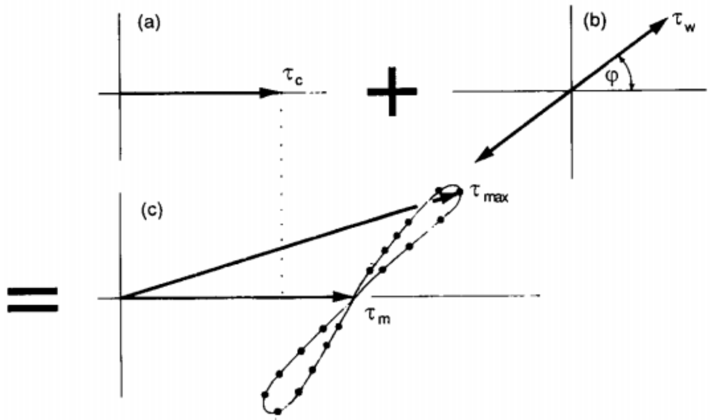

Figure 1: Wave-Current Interaction (SOULSBY
The operating principle of the wave-current interaction

As shown in Fig. 1, in wave-current interaction, two parameters are important for sediment transport: it is the resultant maximum stress $\vec{\tau}_{max}$ and the resulting mean time shear stress $\vec{\tau}_m$. In fact, the sediment movement initiation at the bottom is determined by $\vec{\tau}_{max}$ while the current velocity and the diffusion of suspended sediments are determined by $\vec{\tau}_m$. If we write $\vec{\tau}_t = \vec{\tau}_w + \vec{\tau}_c$, the modulus of $\vec{\tau}_t$ is given by:

$$\|\vec{\tau}_t\| = [\|\vec{\tau}_w\|^2 + \|\vec{\tau}_c\|^2 + 2\|\vec{\tau}_w\|\|\vec{\tau}_c\| \cos\varphi]^{1/2} \tag{1}$$

Is quite clear that the computation of $\|\vec{\tau}_t\|$ requires the knowledge of $\|\vec{\tau}_w\|$ and $\|\vec{\tau}_c\|$ separately.

2.2 Shear stress rate due to wake waves

For a wave propagating with an amplitude H_m and a period T_w, the modulus of the shear stress $\|\vec{\tau}_w\|$ is given by:

$$\|\vec{\tau}_w\| = \frac{1}{2}\rho f_w U_0^2 \tag{2}$$

Where f_w is the friction coefficient of the wave, U_0 is the orbital velocity of the wave. In the case of shallow waters, orbital velocity is given by:

$$U_0 = \frac{H_m \omega}{2\sinh(kh)} \tag{3}$$

In the wake waves situation, the amplitude of the wave is estimated according to the characteristics of the boat as:

$$H_m = 0.5\frac{V_b^{8/3}}{g^{4/3} y_s^{1/3}} \tag{4}$$

Where V_b is the boat velocity and y_s is the distance from the boat midline to the bank (called also sailing length). The average period T_w of the wake waves is expressed as a function of the celerity wave C by:

$$T_w = \frac{2\pi C}{g} \quad ; \quad C = V_b \cos\theta \tag{5}$$

Where θ is the direction of the wake waves. It is possible to calculate the characteristics of the wake waves by considering the linear theory of the wave propagation and assuming that the flow is potential.
In addition, if we assume the flow is in shallow water mode, the dispersion equation is given by

$$\omega^2 = gk\, tanh(kh) \tag{6}$$

Where h is the depth, ω is the oscillations frequency and $k^2 = k_x^2 + k_y^2$ is the wave number.

To determine U_0, it is necessary to estimate the wave number *k* from the dispersion (6). The calculation of *k* can use any iterative numerical method. In this study the Newton-Raphson method was adopted and converged in few iterations.

For the coefficient of friction f_w we adapt the empirical expression given by SIGNELL, R.P. et al. (1990). It expresses f_w according to the roughness $k_b = 30\, z_0$ and the excursion of the wave $A_b = U_0/\omega$ as:

$$\begin{cases} f_w = 0.13(k_b/A_b)^{0.4} & \text{if } \; k_b/A_b < 0.08 \\ f_w = 0.23(k_b/A_b)^{0.62} & \text{if } \; k_b/A_b < 0.08 \\ f_w = 0.23 & \text{if } \; k_b/A_b \geq 1 \end{cases} \quad (7)$$

Finally, t this stage, we calculated all parameters involved in the calculation of $\|\vec{\tau}_w\|$ given by the relation (2).

2.3 Shear stress rate due to current

In addition to the flow velocity, the boat displacement induces a new velocity called return current, denoted U_R (Fig. 2). Depending on the direction of movement of the boat, this current will be added to or subtracted from the river current designed by U_C. Therefore, the stream resulting current noted U_{CR} is given by:

$$\begin{cases} U_{CR} = U_C + U_R & \mathit{if} \text{ the boat riding the river} \\ U_{CR} = U_C - U_R & \mathit{if} \text{ the boat descending the river} \end{cases} \quad (8)$$

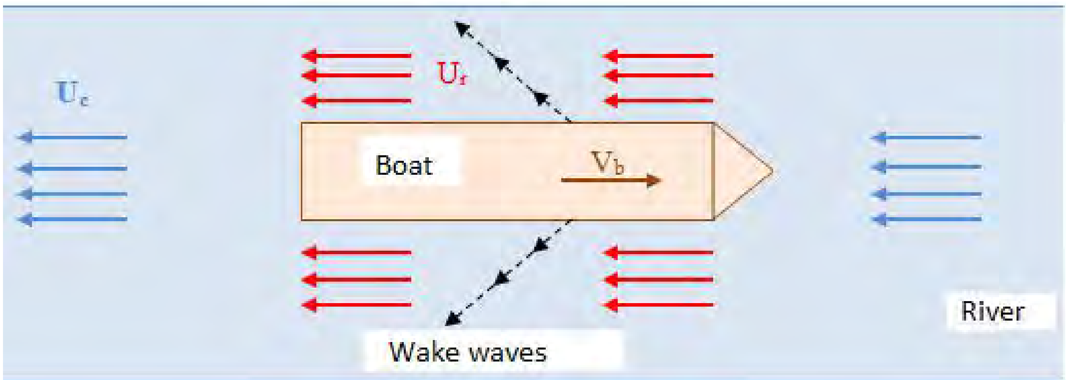

Figure 2: Schematic wake waves
Designation of different current involving on the boat hydrodynamic.

On the other hand, the river current U_C may be expressed as a function of the flow discharge of the river Q_{ru} and the section $B_r h_r$ (B_r being the width of the river and h_r is the height of water):

$$U_c = \frac{Q_{ru}}{B_r h_r} \quad (9)$$

In this work, we adopt the return current U_R estimated empirically according to boat characteristics by the formula established by MAYNORD, S. T (1996):

$$U_r = V_b \times 0.16 \times \left(\frac{B_s}{h_r}\right)^{0.54} \times \left(\frac{T_e}{h_r}\right)^{0.68} \tag{10}$$

Where T_e designates the draft of the boat and B_s its width. To include the return velocity, $\|\vec{\tau}_c\|$ must be expressed by U_{CR} instead U_C . Finally $\|\vec{\tau}_c\|$ should be computed by the following relation:

$$\|\vec{\tau}_c\| = \frac{1}{2}\rho f_c U_{cr}^2 \tag{11}$$

Where U_{CR} resultant velocity and f_c is the current friction coefficient of friction that can be estimated from the Chezy, Strickler or Manning formulae. In this paper, the constant value of f_c equal to 2.5×10^{-3} seems a suitable value of our flow conditions.

In sum, at this stage, we have both an estimation of $\|\vec{\tau}_w\|$ and $\|\vec{\tau}_c\|$. Therefore, we can easily calculate $\|\vec{\tau}_t\|$ by the formula (1). Consideration of river traffic then consists in replacing in the calculation of the erosion the shear stress due to fluvial current by the resulting shear stress given by (1) that include U_R(return current) , U_C(fluvial current) and U_0(wake wave current).

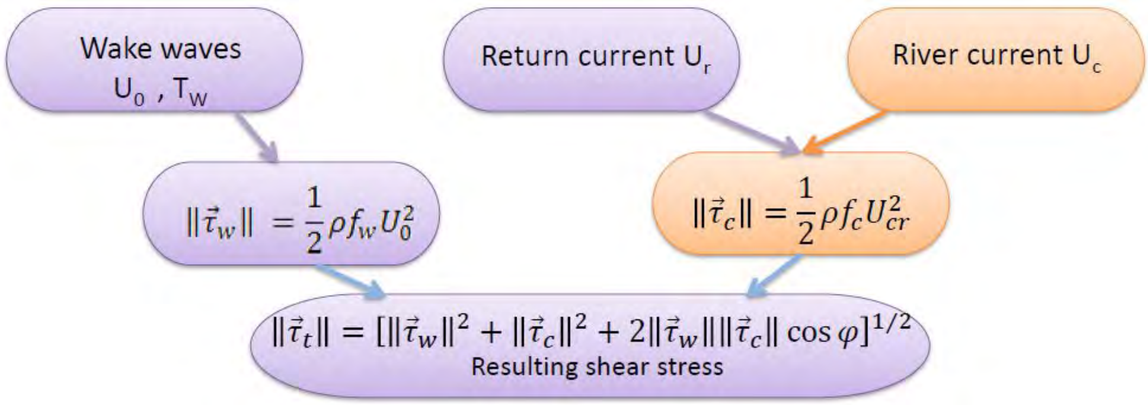

Figure 3: Computation of the total shear stress
Integration of the return current (U_R) , the fluvial current (U_C) and wave current (U_0) in computation of total shear stress ($\vec{\tau}_t$)

It should be noted that it can be shown that in the Kelvin wake wave system, the transverse waves propagate at an angle of 19.47 degrees (relative to the line of navigation). This mean that $\varphi = 19.47^0$. In reality, for the calculation of the total shear stress $\vec{\tau}_t$, the value of this angle depends on the direction of navigation: for boat riding the river, $\varphi = 180 - 19.47^0$, for boat descending the river and if $|U_c| > |U_r|$ then $\varphi = 180 - 19.47^0$. If $|U_c| < |U_r|$ then $\varphi = 19.47^0$.

In order to estimate the lateral erosion distance ΔW, we follow the algorithm proposed by OSMAN , A.M. and THORNE, C.R. (1988). This algorithm is described in the following steps:

1) given de critical shear stress τ_{crit} and γ_w the specific weight of water;
2) compute shear stress τ_t due to the fluvial flow (output numerical model);

3) Determine the initial rate of soil erosion R from: $R = 223 \times 10^{-4}\tau_{crit}e^{-0.13\tau_{crit}}$

4) The initial lateral bank erosion rate is given by: : $dB = R/\gamma_w$

5) The rate of soil erosion, R, is assumed to have an approximately linear increase with shear stress once the critical shear stress is surpassed. Consequently, the actual erosion rate, dW, is

$$dW = dB \times \left(\frac{\tau_t - \tau_{crit}}{\tau_{crit}}\right) \tag{12}$$

6) and if the duration of the flow shear, τ_t, is Δt, then the lateral erosion distance during this time is:

$$\Delta W = dW \times \Delta t \tag{13}$$

3. APPLICATION TO THE REALISTIC CASE

In this section, we try to treat a real case: This is the erosion of a bank portion of the Oise river. For this site, we have bathymetric data as well as flow data and water heights. This data will be needed as the value at the open boundaries of the numerical model. For the hydrodynamic calculation, our choice fell on the open source code "Delft3D" for its ease of use and its robustness. The Delft3D code, does not directly address bank erosion, but has some numerical tools that will be coupled with our calculation of bank erosion to reach our expectations.

3.1 Presentation of the Delft3D code

The Delft3D code is a numerical model that solve the Navier-Stokes 3D equations under classical Boussinesq assumptions and hydrostatic pressure. It also solves a series of convective-diffusive transport equations of a scalar that can represent temperature, salinity or a passive tracer. The Delft3D model incorporates a sediment transport module for morphodynamics to simulate bottom change under wave-current flow. In the same way it integrates a module for the study of water quality. Delft3D simulates the flow and sediment transport in coastal areas, lakes, estuaries and rivers. This code allows the calculation of the flows due to atmospheric pressure gradients; to the wind; to the gradients of salinity and / or temperature and due to the tidal propagation. The Delft3D code also solves the ocean-atmosphere coupling and wave-current coupling flows by using SWAN code for wave propagation. For river applications, the Delft3D code allows the calculation of flows through hydraulic structures such as dams, weirs, etc.

Delft3D takes into account the topographical variations of the studied site by using the σ-coordinate transformation. It takes into account the effects of the Earth's rotation represented by the Coriolis forces. In shallow waters, the exploitation of the long wave characteristic allows the Delft3D model to separate the calculation into two modes: external mode (or barotropic mode) and internal mode (or baroclinic mode). The barotropic mode is obtained by integrating the Navier-Stokes 3D equations on the vertical and assuming the hydrostatic pressure hypothesis. The baroclinic mode is obtained by difference between the Navier-Stokes 3D equations and the Saint-Venant equation. This mode calculates the vertical contribution of each variable of the flow. As for the effects of turbulence in the Delft3D code, they are calculated by choosing a turbulence model from the two-equation models $k - l$ and $k - \varepsilon$ or the LES (Large Eddy Simulation) models. Finally, the high resolution schemes adopted for the convective terms allow Delft3D to simulate correctly the secondary flows in meander curvatures and therefore correctly simulate sediment transport.

To discretise correctly the complex geometry of the studied domains, Delft3D considers the equations of the model in an orthogonal-curvilinear coordinate system. In the horizontal plane, the Delft3D code supports two coordinate systems: the Cartesian system (ξ, η) and the spherical system (λ, ϕ). The equations of the model are discretized by a finite differences scheme in space and the implicit scheme of Crank-Nicolson in time. For stability reasons, Delft3D uses the arrangement of variables on the C-Arakawa grid. This type of grid consists of shifting the variables so that the water level (or the pressure) is calculated at the center of the cell while the components of the velocity are calculated on the facets of the cell. Finally, the resolution of the linear system resulting from the spatial-temporal discretization of the Delft3D numerical model equations adopts the alternating direction implicit techniques (ADI).

3.2 Flow of Oise river

In this part, we study the bank erosion of a portion of the Oise. The choice of this application site was guided by the availability of 222 days data concerning bathymetry, discharge and water level. But also because the Oise is the workshop site of our laboratory (LHN) since 2006. The simulations undertaken on the Oise integrate the effects of river traffic. This integration was carried out by coupling the Delft3D code and erosion module developed by our self. Indeed, the Delft3D code provides the shear stress due to fluvial flow $(\vec{\tau}_c)$, while our modeling provides the shear stress due to the passage of a boat $(\vec{\tau}_w)$. The modulus of the vector sum of these two constraints $\|\vec{\tau}_t = \vec{\tau}_w + \vec{\tau}_c\|$ given by the expression (1) which intervenes to calculate the withdrawal of the bank (ΔW). It should be noted that we only study the erodible part of the bank.

The coupling was done in two stages:

1) We performed the hydrodynamic simulation by Delft3D for the discretized domain of 16 meshes in the direction ξ and 155 meshes in the direction η. The total time simulation is 266 days with a time step of $\Delta t = 30\ s$. At the upstream we imposed a discharge time series considered constant over the width of the river, while at the downstream, we imposed a water level time series assumed constant across the width. During the total time simulation and at a few points along the bank, we extracted the values of the shear stress $(\vec{\tau}_c)$.

2) For a given boat, we calculate the return current and the characteristics of the wake wave generated by this boat (the orbital velocity U_0 by the expression (3) and the period T_w according to the relation (5)). The shear stress due to the river flow $(\vec{\tau}_c)$ calculated by the Delft3D code was read by our code to which we added (or subtracted) the shear stress due to the return current. Our code also calculates the shear stress $(\vec{\tau}_w)$ due to the wake wave impacting the portion of the bank. At this stage, we have been able to evaluate the shear stress $\|\vec{\tau}_t\|$ due to the combination of the fluvial flow and wake wave interaction and therefore calculate the withdrawal of the bank (ΔW) in the presence of river traffic.

The study area is located at the south of the city of Compiègne (France). It extends over a length of 7 km with portions of erodible banks (Fig. 4). For this area we have water level measurements at downstream and flow discharge measurements at upstream over a 266-day covering correctly of high and low water periods (Fig. 5). It should be noted that the downstream limit of the numerical model has been located at the "Venette" dam for which we have water level measurements between 3.0 and 3.5 m to regulate flow.

Numerical simulations showed a spatial variation of the shear stress computed by the Delft3D code at two different times (see IBRAHIM, M & SMAOUI, H (2016)). During these two different moments, we observe a heterogeneous spatial variation. This heterogeneity is undoubtedly due to both the geometry of the river and the variation of the bathymetry. It can be seen that the maximum shear stress is about $2.5\ N.m^{-2}$ and is reached during the 38th day. Similarly, this constraint stress is around $0.45\ N.m^{-2}$ on the 222th day, which is 4.5 times less than the 38th day. These results are in perfect agreement with the hydrograph of this portion of the Oise which presents a flood on the 38th day and a low water level on the 222th day (see IBRAHIM, M & SMAOUI, H (2016)).These results also show that the Delft3D code simulations respond correctly to the demands of a river by reproducing its hydrograph.

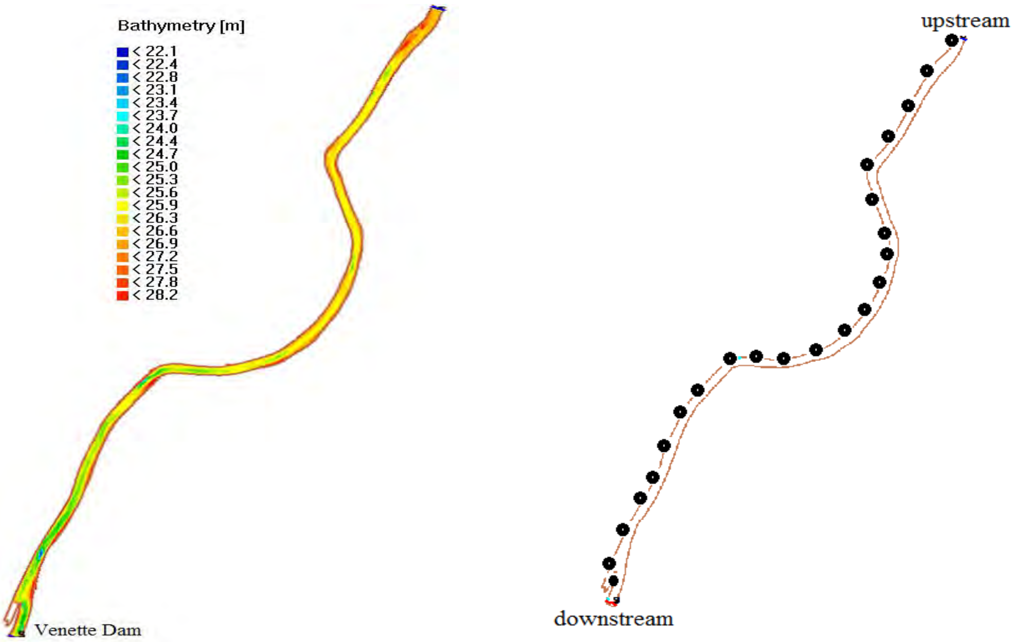

Figure 4: Studied area
Left: the bathymetry in IGN reference; Right: The extraction points of the near bank shear stress.

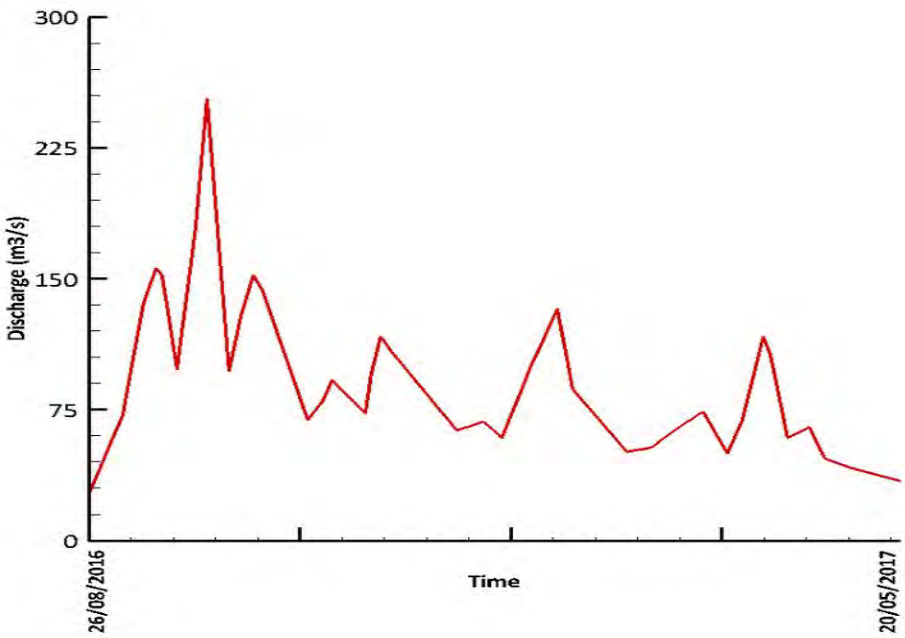

Figure 5: hydrograph
Hydrograph at upstream. of the studied area.

3.3 Estimation of erosion of the left bank

The results of the simulations that will be presented later in this document were obtained by simulating the passage of a boat of width $B_s = 2.5$ m, draft $T_e = 1.0$ m and sailing with a speed

$V_b = 15.0\,\mathrm{km/h}$. It should also be noted that the calculation of the lateral erosion distance ΔW requires knowledge of the critical stress τ_{crit} characterizing the bank soil constituting . As far as we are concerned, we do not have this information for the type of banks of the Oise considered in this study. We have adopted the value of $\tau_{crit} = 0.4\ Nm^{-2}$ corresponding to a clay soil that seems close to the soil of the portion of the bank studied.

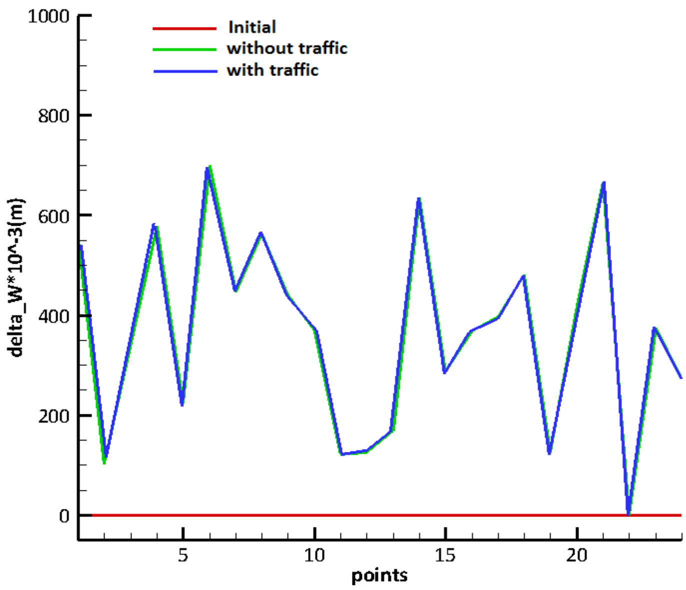

Figure 6: Lateral erosion
Lateral erosion on selected points on left bank..

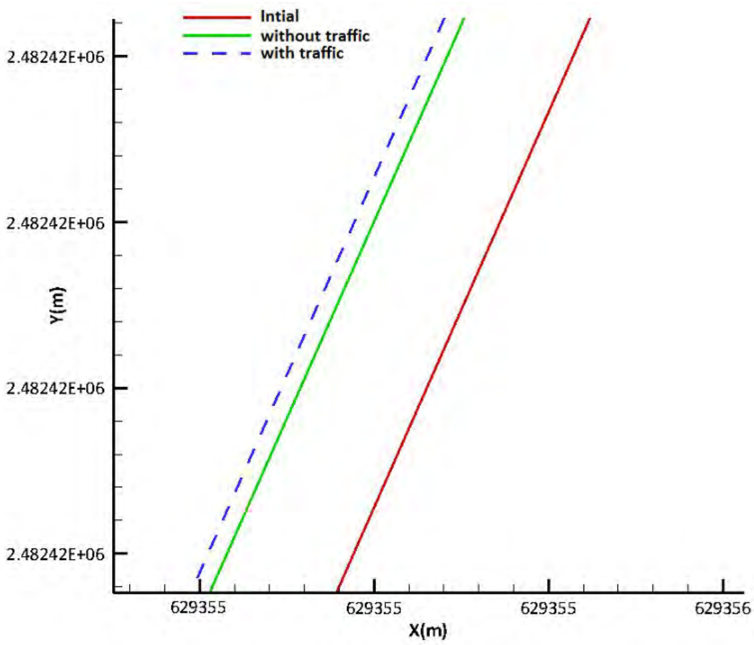

Figure 7: Lateral erosion
Zoom on bank portion

For simulations with fluvial traffic, we present in figure (6) the lateral erosion during 222 days in 24 points extracted along the left bank (Fig 5). This figure superimposes lateral erosion with and without a boat. In this figure, we can not distinguish also the eroded profile without boat from the one eroded by the boat pass. To appreciate the difference between the two profiles, figure (7) illustrates an enlargement of a portion of the bank (between points 12 and 13) which clearly shows the part of the erosion of each sollitation (with boat or without boat) compared to the initial profile. An analysis of Figure (7) shows that during the 222 days, this portion was eroded in total $\Delta\mathrm{W} = 0.63\ \mathrm{m}$. The part of erosion without a boat (fluvial erosion) is $\Delta\mathrm{W} = 0.58\ \mathrm{m}$ from which we deduce that the part of erosion due solely to river traffic is $\Delta\mathrm{W}_{\mathrm{rt}} = 0.05\ \mathrm{m}$. In absolute quantity, the contribution of river traffic is only about 8%, while the erosion due to the flow is very important and accounts for 92% of the total erosion. It should be noted that the value of $\Delta\mathrm{W}$ were estimated from the schematic value of τ_{crit} . The expression (12) shows that the value of $\Delta\mathrm{W}$ depends on the excess of shear stress ($\tau_t - \tau_{crit}$), therefore the accuracy of $\Delta\mathrm{W}$ depends on the accurate knowledge of τ_{crit}.

4. CONCLUSION

During this work, we proposed a numerical model for estimating the shear stress due to the interaction between a fluvial flow and a flow generated by the passage of boats (return current and wake waves). The coupling algorithm of these different shear stress was inspired by the work on the wave-current interaction in the marine environment. The application of the developed model to the real case of the Oise river has shown that the lateral erosion due to both the return current and to the wake of waves is much less important than that caused by the fluvial flow. Note that in this work we have announced 8% as the part of the erosion due to the passage of boat, in fact this value is to be taken with precaution, because of the lack of information on the erosion critical stress of the bank soil, we adopted the value of $0.4\ Nm^{-2}$. This value corresponds to a clay soil that seems close to the soil nature of the studied bank portion !!

It should also be noted that the results of this numerical modeling are preliminary, since it is the first attempt at quantifying bank erosion by river transport undertaken at the LHN laboratory. It is therefore clear that the model we have proposed is only the beginning of a long process of development that will lead to a 3D modeling. On the other hand, we retain from this study that bank erosion is mainly due to fluvial flow. But we also think that the stability of the banks also contributes to their erosion. For that, we intend to develop a modeling coupling the fluvial flow, passage of the boats and the flow in the aquifer juxtaposing the studied bank. Indeed, the safety coefficient F_s is defined generally in its reduced form. In reality, this coefficient depends on the parameters of the flow in the aquifer as an unsaturated medium (pressure in the air pores $\mathrm{P_a}$ and the pressure in the pores fluid $\mathrm{P_f}$). It is for this reason that the development of a model of the flows in unsaturated medium is necessary to have access to the parameters $\mathrm{P_a}$ and $\mathrm{P_f}$ which appear explicitly in the expression of the safety coefficient $\mathrm{F_s}$. Finally, we will also consider coupling with a bank failure model.

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6. COPYRIGHT

Multifunction of Inland Waterways – Social and Environmental Awareness of IW Managers

by

***Andreas Dohms*[1], *Tom Denes*[2], Marc Demanet[3] *and Yvon Loyaerts*[4]**

Full Paper

1. INTRODUCTION

The topic of the new Working Group "Sustainable Inland Waterways – a Guide for Inland Waterway (IW) Managers on Social and Environmental Impacts" (INCOM WG 203) refers to the opportunities and challenges for IW managers, stemming from multiple uses and functions of inland waterways. It also refers to the social and environmental awareness of the managers who are responsible for operating and developing those waterways. The objective will be to provide a general guiding document for IW managers to increase their awareness of the need for developing more sustainable inland waterways, considering their uses and functions being represented by stakeholders such as communities, organizations and people, but also by industry and all kinds of businesses. By studying earlier reports or concurrent documents the working group will work out how the approach will impact the management of inland waterways:

The topics for the session's presentations are:

-	Values and uses on inland waterways	by **Andreas Dohms**
-	What is social and environmental awareness of managers, also called: Corporate Social Responsibility (CSR) ?	by **Tom Denes** and **Marc Demanet**
-	Why do decision makers and engineers need CSR and a multifunctional approach? Some applications	by **Yvon Loyaerts**

2. Values and Uses on Inland Waterways

2.1 Introduction

Human life has always been connected to water. Besides providing direct water supply, the use of the rivers as a transport mode for cargo and people was another very important motivator for settling there. In former times inland navigation was the only transport mode that was both available and secure. By connecting the rivers with artificial canals, waterway systems have been created and developed.

Still today, inland navigation remains an important part of freight transportation, nationally and internationally. Furthermore, inland navigation is recognized as an environmentally friendly transport system, in particular in terms of its low specific fuel consumption resulting in less carbon dioxide and exhaust gas emissions in comparison to other modes of transports.

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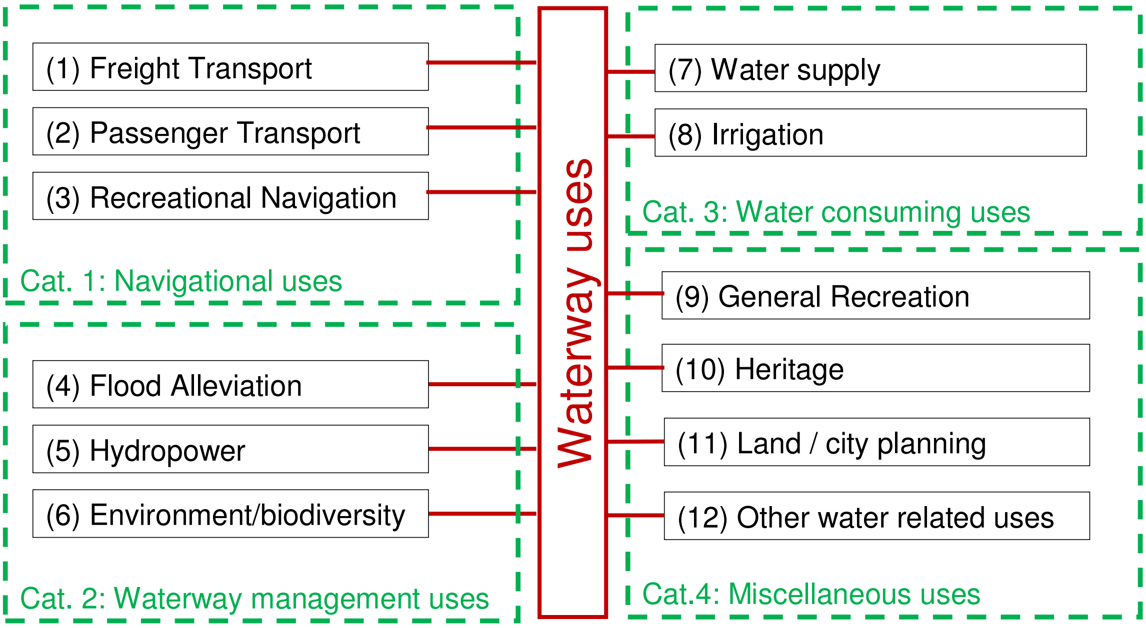
Inland waterways were and are veins for freight transports, and contribute to national economies. But, Inland waterways are more than that. Rivers and canals fulfill many other functions, being important for the society as well. To operate and develop those waterways, IW managers are required to consider a wide range of interests coming from the multiple functions of waterways.

2.2 Structuring the values or uses of inland waterways

Referring to previous investigations, in and outside PIANC, inland waterways are more than just means for transportation. By the former InCom working group 139 "Values of inland waterways" (report released in 2016), 12 principal uses of inland waterways are raised:



This list gives an impressive view on how waterways can be useful or valuable, not only for the navigational business, but also for a wide range of services to society. This is on the one hand, an advantage of inland waterways in comparison to other means of transports. But on the other hand, it is a challenge for waterways organisations, which are responsible for managing, operating and developing those waterways. In addition, waterways are confronted with a wide range of interests, represented by many stakeholders that are inhomogeneous to each other.

2.3 Analysis of the values or uses of inland waterways

The evaluation of the uses, as mentioned above, considers their use and non-use benefits, complemented by physical and managerial aspects as well as mutual interactions with other uses. So, the working group devised seven key aspects, listed below:

Characteristics and management of the waterway	
1	Physical aspects of the waterway
2	Operational aspects
Socio-economic benefits and environmental impacts	
3	Economic aspects
4	Environmental aspects
5	Social aspects

Differential considerations	
6	Interaction
7	Balance of interest

<u>What do the key aspects stand for?</u>

Physical aspects of the waterways – pertain to physical waterway characteristics and structures and the hydraulic conditions

Operational aspects – pertain to the waterway's operation such as safety issues, traffic rules, operational times, information systems, etc.

Economic aspects – pertain to the economic costs and benefits of all waterway uses

Environmental aspects- pertain to positive or negative effects on the environment such as impacts climate, flora and fauna, habitats in general, protected areas, etc.

Social aspects – pertain to effects on social conditions and well-being of humans in areas such as employment, safety, health, social cohesion etc.

Interaction – pertain to the mutual influence of benefits from various waterway uses – reinforcing, weakening or conflicting to each other.

Balance of interests – pertain to the consideration of certain uses preferred above other possible uses – including the interest of the various stakeholders.

The above mentioned aspects have been used as a checklist for evaluating the different waterway uses.

2.4 Describing and evaluating one waterway use: Recreational navigation

An example of the methodology of describing and evaluating waterway use for recreational navigation is shown below:

Recreational navigation covers a wide variety of water related activities such as boating, yachting, sailing, canoeing, surfing, fishing, rafting, and kiting. Passenger shipping, river cruising or operating ferries can be referred to recreational navigation as well in case they are for touristic purposes.
Sailing along waterways for one's own pleasure, whether individually or in groups, has become an important part of the lifestyle in many countries, both developed and in developing countries. The scale of recreational navigation has largely increased during the last few decades, thus increasing an important economic sector on its own.
Recreational navigation is performed along large rivers or canals (in Europe i.e. on the Rivers Rhine, Danube or Rhone), which are often heavily used for freight transportation. Recreational navigation is also performed along smaller waterways, which often had been originally been used for transport purposes during the first industrial revolution (18^{th} or 19^{th} century), but nowadays without any freight transport. So, developing recreational navigation became important for conservation, but also for a revival of some of those historic waterways. Examples are the broad canal network in the UK, used for the so-called narrowboats, as well as the Erie-Canal in the United States and the Göta-Canal in Sweden.
In summary, recreational navigation is suitable for a wide variety of water related activities that offer opportunities for the waterways and the surrounding areas. But it is important to note that investments in waterways for recreational purposes or for supplemental landside infrastructure cause impacts that can result in inconsistent interests. This can be evaluated using the key aspects as follows:

Physical aspects of waterways: Large waterways with freight transport can be used for recreational navigation as well. Additional equipment for the smaller boats might be necessary. Impacts on the surrounding environment will be low. For small waterways, investments with construction related changes on size or shape of the affected channels might be necessary. Impacts on the surrounding environment will be inevitable, probably by establishing a technical designed monotone waterway.

Operational aspects (Figures 1 and 2)**:** on large waterways, recreational navigation is an additional use. For that, extra equipment or the separation of traffics for safety reason might be necessary. On smaller waterways, without fright transports, recreational navigation requires the complete operational equipment, like locks, i.e. self-served locks with all signal and signs, as necessary. The level of the provided equipment and the reliability or this waterway is essential for the attractiveness of it.

Figure 1 (left): Commercial vessel and recreational boats in the same lock chamber
Figure 2 (right): Self-service lock a an small waterway used only for recreation

Economic and social aspects: recreational navigation is suitable to create businesses such as boat-related businesses (constructions and repair) or tourism (hotels, restaurants, camping site, shopping areas) and thus support economic development and social well-being by creating jobs. Historic waterways that have been neglected can be re-constructed and made fit for navigation. The whole surrounding benefits.

Environmental aspects: the natural and the human environment are affected by recreational navigation. Construction work for developing the waterway has impacts on the direct surroundings. The navigation itself causes exhaust gas and noise emissions. When such waterways go through protected areas, the impact on them becomes direct.

Interaction: Basically, the better the navigation conditions—by maintaining and making improvements—the more navigation will be attracted. So, these aspects reinforce each other. As shown above, recreational navigation and economic development have great potential to reinforce each other. But on the other hand, recreational navigation and the environment have the potential to conflict each other. However, by using the environmental potential—for example for touristic purposes—sustainable development of the waterway and the surrounding region is highly possible.

Balance of interests: benefits from recreational navigation can be created, but there may be impacts on people or on the environment. Thus, the interests in this matter, represented by stakeholders, can be different, even contradictory. Communities and business organization often support navigational activities, hoping for economic and social development. Environmental organizations, activist groups or individuals might have different interests.

Finding a real balance of those potential interests is essential for developing waterway projects successfully. In addition, considering all these interests from the beginning is necessary to develop waterways in a sustainable way.

The next two chapters show tools that can be used and why a multifunctional approach is the best approach for waterway managers or decision makers.

3. What is Social and Environmental Awareness of Managers?

The American philosopher and scientist Alfred Korzybski said that "a map is not the territory it represents ». We may easily apply this thinking to the concept of "Sustainable development" and "Corporate Social Responsibility" as it can cover different understanding of one person to another. Moreover, the words that make up these concepts "sustainable", "Corporate" and even "Responsibility" doesn't necessarily have the same meaning for every person and are strongly linked to the place where they live (education, historical development) and the main economic system surrounding their day to day love of work.

To prevent as far as possible a misunderstanding and as an introduction of this paper, it is important, at first, to provide the reader with a clear definition of these concepts and words. This will provide the reader with the authors' understanding and vision about this large subject, as applied to the inland waterways management.

3.1 Introduction of the Concepts Used in this Document

Following the definition provided by ISO26000 [1]:

- **Sustainable development :** "***development that meets the needs of the present without compromising the ability of future generations to meet their own needs***"

- **Social responsibility :** " ***responsibility of an organization for the impacts (positive or negative) of its decisions and activities on society and the environment, through transparent and ethical behaviour that :***
  – contributes to sustainable development, including health and the welfare of society;
  – takes into account the expectations of stakeholders (individual or group that has an interest in any decision or activity of an organization);
  – is in compliance with applicable law and consistent with international norms of behaviour; and
  – is integrated throughout the organization (entity or group of people and facilities with an arrangement of responsibilities, authorities and relationships and identifiable objectives) and practised in its relationships "

  In accordance with this definition, the UE Commission has defined Corporate Social Responsibility (CSR) as "the responsibility of enterprises for their impact on society" [3]

  The concept of "Corporate Social Responsibility" is composed of three balanced words; each word has its importance :

  - ***Corporate :*** Following the ISO26000 and our understanding, the word "Corporate" covers all organizations (whether private or public) that can have an impact, at their scale, through their actions on a more sustainable development of our society. Further to this definition, the public departments of inland waterways are also included under the word "Corporate" because they have the assignment to oversee and manage the inland waterways and facilities, and also have clearly a huge role in the sustainable development of the inland waterways.

  - ***Social :*** In the context of CSR for companies, it could refer to the relationship between the company and its employees. However the concept of "social" is much larger. Here, the word "social" must be understood as a concept that includes all social dimensions related to sustainable development of the world surrounding us.

- ***Responsibility :*** It's necessary to make the distinction to the "Latin" and the Anglo-saxon economical and cultural approach. For the "Anglo-saxonne" approach", the word "Responsibility" is more related to "accountability". In this meaning, CSR is seen in a restricted point of view, as the companies are only accountable to the owner of the company. Concerning the "Latin" approach, the word "responsibility" may refer to responsibility (as in the sense that it's a non-compulsory commitment: that is the company's commitment to do something for a more sustainable development) or accountability (the obligation for a company to do something and, in case of a problem, be accountable for the consequences). Following the definition of the UE Commission the word "responsibility" has to be understand by :
    - following the law;
    - integrating social, environmental, ethical, consumer, and human rights concerns into their business strategy and operations.

  The UE Commission approach is closer to the "Latin" approach than "the Anglo-saxonne" one. In the present document, we 'll use this definition.

Indeed, the CSR is based on seven ISO 26000 principles :

- ***Accountability:*** an organization should be accountable for its impacts on society, the economy and the environment. However this accountability is limited to:
    - "the impacts of its decisions and activities on society, the environment and the economy, especially significant negative consequences ;
    - the actions taken to prevent repetition of unintended and unforeseen negative impacts" [1]
- ***Transparency:*** "an organization should be transparent in its decisions and activities that impact on society and the environment" [1] It seems obvious that the principle of transparency is one of the bases of the CSR principles as it clearly allows the organisation to show its real commitment to participate, at its scale, in global sustainable development.
- ***Ethical behaviour:*** the ethical behaviour appears in the values promoted by the organisation and its way to live with them-- its governance.
- ***Respect for stakeholder interests:*** "an organization should respect, consider and respond to the interests of its stakeholders". Regarding a public organisation, it means to find the right balance between different interests toward a more sustainable development.
- ***Respect for the rule of law:*** this principle is of course mandatory and obvious for a public organisation in charge of the inland waterways Management.
- ***Respect for international norms of behaviour :*** when the local norms don't exist or when they are below the level of requirements set by them, the organisation should strive to respect such international norms of behaviour to the greatest extent posible.
- ***Respect for human rights :*** this principle is of course mandatory and obvious for a public organisation in charge of the inland waterways Management.

3.2 Relation between Sustainable Development and Corporate Social Responsibility

According to the above definitions, Sustainable Development has to be understood at a global and macro economic level [2] Sustainable development is a global objective that every country should include in their development (which is one of the ideas of the COP 21 in Paris).

The main objective of sustainable development is to achieve global sustainable development of our world. Under this assumption, it doesn't mean that the organisation implementing CSR has the objective to be a sustainable organisation.

Corporate Social Responsibility is located at the level of the organisation (micro economic level). An organisation that implements CSR contributes to global sustainable development because the organisation takes into account the economics, social and environmental consequences of their choices as well as its strategic development and management. Moreover, the organisation, implementing the CSR concepts, shows its awareness of its impacts on society at its scale and shows its willingness to take into account the consequences of its activities on society, as well as its desire to globally orient its actions towards sustainable development.

3.3 Why the Concept of CSR is so Important Nowadays

The notion of sustainable development was gradually built up during the period after the Second World War. During this period, the majority of developed countries experienced strong growth. At the end of the 1960s, a group of scientists (called the Club of Rome) questioned the impact of this growth on the planet and its occupants.

This resulted in 1972 in the Meadows Report titled "The Limits to Growth". On the basis of simulations by computer models, the possible consequences of the evolution of the human population according to the exploitation of the natural resources, were simulated until about 2100. It showed that by 2100, economic growth would result in a sharp fall in population because of pollution, the impoverishment of arable land, or the depletion of fossil fuels.

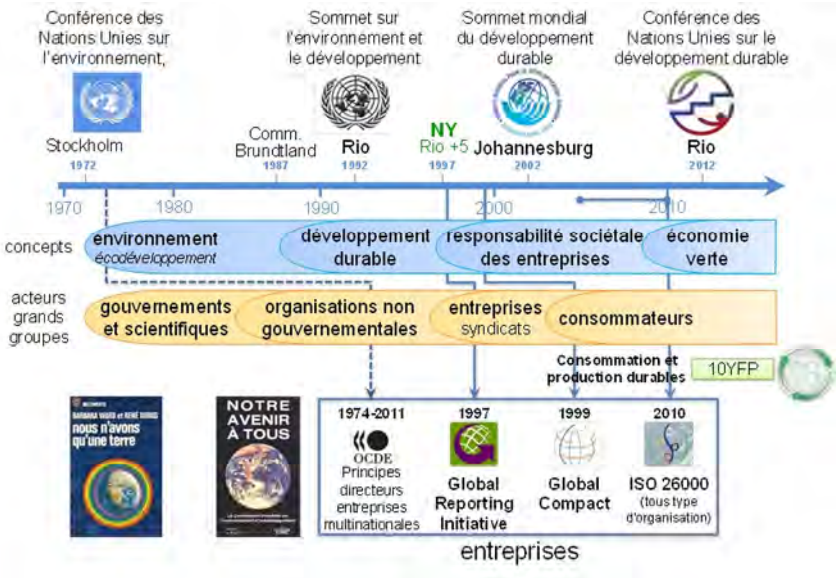
In 1972, the Conference of the United Nations on the Human Environment in Stockholm featured eco-development, the interactions between ecology and the economy, the development of the countries of the South and North. This was the first step towards a more integrated vision economy-living environment.

In 1987, the World Commission on Environment and Development (Brundtland report) proposed a definition of "sustainable development"; the definition mentioned above. This report shows that to think of a "sustainable" future, we must act in the present.

In 1992, the third Earth Summit in Rio de Janeiro gave birth to Agenda 21 (21 measures that states commitment to implement sustainable development). The definition of sustainable development incorporated "three pillars" that need to be reconciled: economic progress, social justice, and environmental preservation.

2005: This is the entry into force of the Kyoto Protocol regarding the reduction of greenhouse gas emissions in the European Union and awareness of the effects of locally produced gases on the planet.

2015: The Cop 21 in Paris set goals to limit the rise in temperature of the planet to preserve future generations and their living environment. This conference was a success because for the first time, the result wasn't a measurement that the different countries had to meet. This time, each country had to propose its contribution to more sustainable development and to limit climate change (the global increasing of the planet temperature due to the consumption of fossil energy.)



In parallel with this historical evolution, we notice a global awareness (that the actions of some impact those of others and that sustainable development must be understood on a global scale). This global awareness, which was initiated within a group of some scientists (the Club of Rome), manifested first to large international organizations and gradually reached governments, companies (through CSR) and finally to individuals.

For companies, public or private, this is reflected in the concept of "societal and environmental responsibility (CSR)" of companies in the broad sense; a concept that is becoming more and more important. In other words, the question that arises for them is how companies, public or private, act at their scale in a global process (economic, societal, and environmental) of a more "sustainable" development.

In summary, the objective of implanting the concept of social responsibility in an organisation is to contribute, at their scale, to a better global sustainable development.

Beyond the expected results of setting up a CSR process in an organisation, the process of awareness raising up by the CSR approach consists in a fantastic motor allowing huge changes in the organization in the direction of a more sustainable development.

4. Why do decision makers and engineers need CSR and a multifunctional approach?

4.1 Corporate approach within public departments in charge of inland waterways

Guarantor of the common welfare, authorities may have a role of example even precursor when speaking about corporate approach. Concerned in essence of the interest of all, they translate in many countries this awareness at the level of the legislations, the societal orientations that they impulse) and through the way they realize it.
Moreover in contrast with other parts of the public administration (like the department of justice for instance), the department in charge of inland waterways can have a direct impact on the sustainable development. By their choices and actions, managers of the inland waterways have a central role in a more sustainable development. Taking into account the different interests of society in its entirety

and at a long-term scale, managers of the inland waterways have the capacity to orient the applied solution in a more sustainable way: for instance, a new dock is no longer seen as only an economical tool but takes also into account environmental and social impacts on the society on a local and global scale.

A new way of thinking and working is taking into account the values IW have and their consequences on technical, economic, environmental and social aspects as mentioned above.

Under this assumption, the department in charge of the inland waterways, shall be covered by the concept of “Corporate”.

More and more IW departments integrate indeed the three pillars of CSR (social, environmental and economic) in their projects and operations. Although being strong this trend is nevertheless not present everywhere and some authorities are still active on a traditional technical way.

Some examples on how engineers and decision makers are taking into account a multifunctional approach in their managerial duties are given below.

4.2 Some examples of corporate approach within IW department

All the examples that follow are taken within recent projects managed by the public authority in charge of the Walloon inland waterways (the “Service Public de Wallonie”).

Wallonia is the southern part of Belgium that is a federal kingdom. Due to the structure of the Belgian State the decentralized authorities have the full responsibility of managing and operating the IW network i.e. for Wallonia 451 km of navigable rivers and canals with 80 locks sites (some having more than one lock), 5 boat lifts and one inclined plane.

Most of the network is rated class IV but the local government approved a major investment plan to upgrade the network to class Va or more (the Meuse river up to class VI and some other sections to class Vb). This global upgrading plan started around 2005 and should be finalized around 2025. The whole network (class IV and above) is included within the Trans-European Network for Transport (TEN-T) and more especially into the multimodal corridor North Sea – Mediterranean. In the western part of Wallonia (mainly related to the Scheldt basin) the projects are integrated in the global European IW project Seine – Scheldt aiming at improving the connection between the Seine basin and the North of France and further with Belgium and the Netherlands.

Three projects are shown as interesting examples:

- The upgrading of the Scheldt river through city Tournai (class IV to Va)
- The building of a marina in the center of city Charleroi as part of a global urban project
- The rehabilitation of the quays alongside the river Meuse in city Dinant

4.2.1 The Scheldt river in Tournai

The Scheldt as it nowadays flows through Tournai is a canalized river that is on some places really narrow making the navigation difficult, uncomfortable and dangerous: under an heritage bridge (width 12 m) or with sharp and narrow curves (down to 19 m width). The need for improvement is obvious given the existing and forecast traffic. Furthermore the river is a real bottleneck on the Seine – Scheldt project.

As the enlargement works are taking place in the historical part of the city, special care was the rule from the first steps of the studies to meet the sensitivity of the inhabitants and of the local politicians. The engineers and decision makers intend from the beginning to embed the project in a global approach taking into account the heritage sites and the environmental and social aspects besides a traditional economic and technical approach. The preliminary studies led to a solution that becomes a global urban project. The reshaping of the river itself was enlarged to improve the surrounding areas taking the opportunity of the water infrastructure works to reshape the surroundings alongside the river. Recreational navigation was not forgotten with specific infrastructure and opportunities for a future marina within a park. Special care was given to the heritage sites (amongst them the emblematic places in the city): several committees were created in order to follow the project and to help define the most appropriate solution. Furthermore the inhabitants were asked to give their advice and preferences through a popular consultation.

The same occurred when writing the tender documents. Such a sensitive project was the opportunity to define an alternative tender process. The price was not the only criteria to choose the contractor. Several criteria were taken into account amongst which the use of the waterway to transport the materials, the daily organization of the works and the care given to limit the discomfort for the inhabitants, the limited consequences on urban mobility, the pollution created by the works or by the related transport, the duration of the work,… During the work specific communication was built up to explain what is going on and the reasons for the works and of additional improvements.


4.2.2 The marina in Charleroi

Charleroi is one of the major historic industrial cities of Wallonia. However the steel crisis and the closing down of the coal mines made its economic situation difficult during the last decades. To foster the local economy again and to bring attractivity back to the city, the local decision makers started with an ambitious plan aiming at reshaping the center of the city through public and private investments.

Several projects are related to the Sambre, a tributary of river Meuse, and its close surroundings. The river flows alongside the center of the city. It is rated class IV and will be upgraded to Va within the next years. In recent years the IW authorities used the occasion of major quay walls maintenance works to rebuild the quays located on the city side: new pedestrian areas were created and the old buildings were refurnished. On the opposite side the railway station area was refurnished with a new plaza, new light rail and bus terminals, multimodal connection center, etc.

The next step is a Public/Private Partnership (PPP) project close to the river that is intended to build up high and medium level apartments. The major element of the project is a marina connected to the river. The marina should be a most attractive point linking still closer the city with its river, fostering tourism and bringing added value to real estate that will develop there.

By doing this the local authorities thanks of the PPP and with the help of the Region enlarge the strategy around the river. The regional IW authority plays an important role: its engineers are supporting the technical aspects and the IW budget is used to promote quality equipment alongside the river.


4.2.3. The quays of river Meuse in Dinant

Dinant is an historic and touristic city alongside the river Meuse close to the French border. Nevertheless, although having a high touristic potential, the city suffered from inadequate old-fashioned equipment alongside the river: the quays were too narrow and uncomfortable and were used by car transit when the terraces were located between the car lane and the old recreational embankments.

Here again the IW authorities took the opportunity for the need of major maintenance work to improve the whole environment alongside the river. The quays were enlarged and overhang now the side of the river. New embankments were built up and the space for the terraces brings now more comfort and attractivity for pedestrians and tourists when the lane is wider and better separated from the terraces area.

The construction phase was a delicate one not only because the works took place in a sensitive urban area: the touristic commodities must be protected during the spring and summer periods. The cooperation between the engineers and technicians (IW authority, regional road authority, facilities managers,…) on the one side and the local politicians and touristic operators on the other side definitely was the key element for a successful project. The city and its mayor were very active players on the ground. The objective of the project gives them the opportunity to promote the city. As an example they use nowadays the name “La Croisette” (as a reference to the well-known Cannes sea promenade) for the reshaped quays along the river Meuse.

4.3. Some final comments

As shown by the examples above a new trend is raising up when the IW authorities are thinking about maintenance or improvement works. The traditional technical approach is nowadays enlarged and translated into a cooperative one where all the stakeholders are taking part to build up consensus and to bring added social value to the projects and finally to the waterway itself.

5. Summary

This paper gives an idea on why operating, maintaining or developing inland waterways in a sustainable way is necessary for IW managers. In the light of the variety of uses or functions, what means of the multifunction of inland waterways, as shown in chapter 2, the broad interest of the society, of organizations or individuals in those waterways is obvious. This awareness requires a comprehensive approach on all kinds of waterway management activities. The concept of "Corporate Social Responsibility", described in chapter 3, provides basics and a guideline how to use it in general. Examples for successful projects, developed in a corporate ways are given in chapter 4.

The task of the new working group will be to use and to adopt all this knowledge for providing a guiding document for IW managers. Experiences from other waterway experts are welcome.

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BERGER A., de PERTHUIS C., PERIN N., (2014), « Le développement durable », Nathan, Paris

7. COPYRIGHT

Congrès international de navigation de l’AIPCN de Panama de mai 2018

« Problématique des choix de priorité et du niveau d’investissement dans la sécurité des ouvrages hydrauliques : le cas du réseau confié en France à VNF **»**

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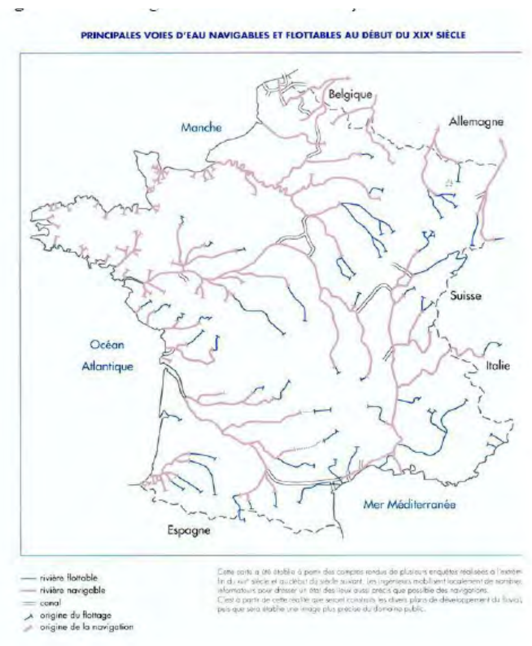
Lors des épisodes de crues du printemps 2016 de la Seine moyenne et de la Loire moyenne, les digues du canal de Briare se sont rompues et Voies Naviagables de France (VNF), autorité en charge de la gestion de ce canal, a été mis en cause, même si sur le fond l’accident n’a pas créé de dégâts considérables. Le Conseil général de l’environnement et du développement durable a diligenté à l’automne 2017 une mission d’audit-conseil [1] pour aider VNF à déterminer ses priorités dans les investissements de sécurité des ouvrages hydrauliques qu’il gère sur les voies navigables dont la gestion lui a été confiée par l’État. Il est apparu utile de pouvoir rendre compte au Congrès de l’AIPCN des principaux enseignements que la mission a retirés de ce travail réalisé entre septembre 2017 et avril 2018, dans la mesure où il est aussi lié aux réflexions internationales de l’AIPCN sur la maintenance des réseaux fluviaux.

Nous nous proposons donc d’aborder rapidement la question de la formation progressive du réseau des voies navigables en France, celle de la réglementation qui s’applique aux ouvrages hydrauliques et celle du niveau de financement de la politique de sécurité en nous référant à quelques exemples similaires, avant de conclure en rappelant trois des principales recommandations formulées par la mission.

<u>1- Formation progressive du réseau fluvial français</u>

Antoine Beyer[2] a retracé assez complètement récemment dans une perspective à la fois historique et géographique la façon dont le réseau des voies navigables s’était établi en France et nous en extrayons les éléments les plus utiles à la compréhension du réseau actuel confié à VNF.



Si les grands fleuves français (Loire, Seine, Rhône, Garonne et Rhin) connaissent la navigation depuis l’époque romaine, le développement des canaux s’est constitué principalement à partir du XVII ème siècle avec le principe des canaux à bief de partage des eaux avec notamment l’ouverture du canal de Briare en 1642, celle du canal d’Orléans en 1687 pour établir la jonction entre Seine et Loire, puis avec l’entreprise magistrale de Pierre Riquet, la réalisation du canal du Midi, dit aussi canal des deux mers, puisqu’il permettait d’opérer la jonction entre la mer Méditerranée et l’océan Atlantique par l’estuaire de la Garonne. Avec l’ouverture du canal du Centre, puis celle du canal du Nivernais sous la Révolution, puis avec l’Empire se dessine un réseau assez complet de voies navigables ou flottables avant l’arrivée du chemin de fer au milieu du XIX ème siècle. La carte ci-contre[3] donne une assez bonne idée de l’extension historique de ce réseau fluvial.

1 Mission d’audit-conseil du CGEDD de MM. Geoffroy Caude,Thierry Galibert et Sylvain Leblanc sur la sécurité des ouvrages hydrauliques de VNF- avril 2018

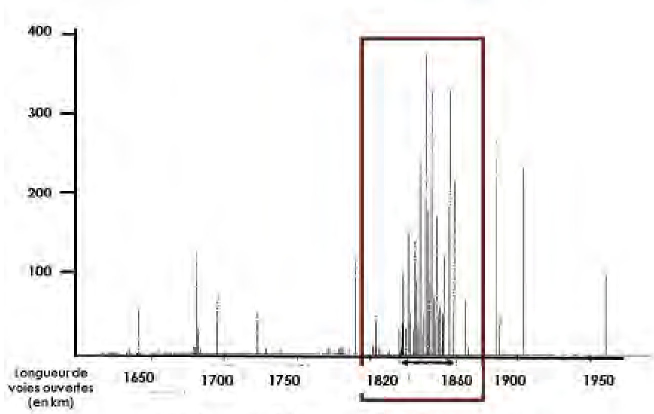
2 Antoine BEYER(2016) : « Les grands jalons de l’histoire des voies navigables françaises ». Pour mémoire, revue des ministères de l’environnement, de l’énergie et de la mer, n°17, été 2016, pp. 83 - 93. En ligne : http://www.developpement-durable.gouv.fr/Dernieres-parutions.html

3 Guy ARBELLOT- Bernard LEPETIT-Jacques BERTRAND- Atlas de la Révolution française- Tome I Routes et communications - Editions de l’Ecole des Hautes Etudes en Sciences Sociales-92 pages

Les canaux y sont mentionnés par deux traits fins noirs parallèles et relient au Nord vers les voies des Provinces Unies, ainsi que les canaux historiques que nous avons déjà mentionnés.

Deux plans vont se succéder au cours du XIX ème siècle, le plan Becquey lancé en 1820 pour accompagner l'industrialisation du Nord-Est dont on saisit en pointillés de couleur rouge les extensions envisagées[4] sur la première carte à gauche et l'extension linéaire entre 1820 et 1860[5] qui figure sur le diagramme à droite, ci-dessous :

Les grandes lignes de navigation préconisées par Becquey (en rouge)



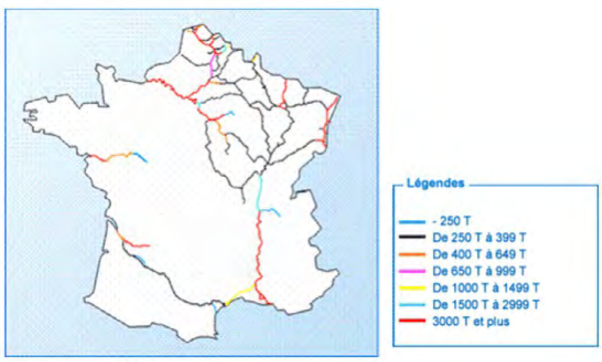
Le boom de la construction des canaux Sous la Monarchie de juillet

La progression du réseau fluvial reprend en 1870 après vingt ans de controverses entre les partisans de l'extension du seul chemin de fer et ceux de la complémentarité avec la voie d'eau en 1870 grâce au Ministre Freycinet qui fige en quelque sorte la dimension standard avec des péniches de 38,50 mètres de long.

Au cours du XX ème siècle, l'extension se poursuit surtout en développant un réseau à grand gabarit (classes 4 à 6), notamment sur le Rhône avec la Compagnie Nationale du Rhône(CNR), sur le Rhin avec la concession d'Electricité de France(EDF), sur la Seine, mais de façon très intermédiaire sur le canal du Nord (classe 3 intermédiaire de 800 tonnes). La desserte proche des grands ports y est améliorée (principalement celle de Marseille, du Havre et de Dunkerque). Le réseau confié à VNF, établissement public qui absorbe en 1991 l'ancien office national de la navigation créé en 1912, est le suivant.

CARTE DES GABARITS DE VOIES NAVIGABLES EN 2003 (source VNF)

Gabarit	Classe	Port en lourd
Grand gabarit	6	+ de 3000 tonnes
	5	de 1500 à 3000 tonnes
	4	de 1000 à 1500 tonnes
Moyen gabarit	3	de 650 à 1000 tonnes
	2	de 400 à 650 tonnes
Petit gabarit	1	de 250 à 400 tonnes
	0	de 50 à 250 tonnes



Source : VNF

Comme on le voit, il comprend environ 1800 kilomètres de réseau à grand gabarit (classe 4 et plus) avec principalement la Seine, l'axe Rhône-Saône, le Rhin, la Moselle jusqu'à Neuves-

4 Charles BERG-2009- Histoire et Patrimoine des rivières et canaux- site internet- http://projetbabel.org/fluvial/presentation_cb.htm

5 Bernard LESUEUR-2012- Navigations intérieures-Histoire de la Batellerie

Maisons et le canal Dunkerque-Valenciennes, le reste étant composé de voies de type Freycinet à quelques exceptions comme celle du canal du Nord.

Le réseau de VNF comprend 6 700 km de canaux, fleuves et rivières canalisées, 4 000 ouvrages[6] et couvre un domaine public de 40 000 hectares.

Il est essentiellement décomposé, selon VNF, en deux réseaux : un réseau fret et un réseau dédié à l'aménagement du territoire.

2- Réglementation applicable en France aux ouvrages hydrauliques

La sécurité des ouvrages hydrauliques est réglementée en France par les dispositions du décret n°2015-526 du 12 mai 2015 qui fait partie du code l'environnement comprenant les dispositions du décret n° 2007-1735 du 11 décembre 2007 mettant en œuvre les dispositions concernant les ouvrages hydrauliques prévue par la loi sur l'eau et les milieux aquatiques du 30 décembre 2006[7].

Les ouvrages sont classés en trois catégories (de A à C selon le niveau de gravité des conséquences possibles de rupture). Ces classes ont été définies principalement pour des barrages(hydroélectriques ou retenues d'eau pour l'agriculture ou pour l'alimentation en eau).

Voici les principes de la classification adoptée :

H est la hauteur de la crête du barrage [8]

V est le volume de la retenue d'eau du barrage [9]

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6 Dont notamment 27 tunnels-canaux, 674 ouvrages de franchissement, 356 barrages de navigation, et 3 756 km de digues.

7 Le nouveau texte crée les notions de »système d'endiguement » et d' « aménagement hydraulique », fait disparaître la classe D, et modifie la périodicité de renouvellement ou de mise à jour de certaines obligations réglementaires :études de danger, rapports de surveillance, d'auscultation.

8 " H " est la hauteur de l'ouvrage exprimée en mètres et définie comme la plus grande hauteur mesurée verticalement entre le sommet de l'ouvrage et le terrain naturel à l'aplomb de ce sommet.

9 " V " est le volume retenu exprimé en millions de mètres cubes et défini comme le volume qui est retenu par le barrage à la cote de retenue normale. Dans le cas des digues de canaux, le volume considéré est celui contenu dans le bief entre deux écluses ou deux ouvrages vannés.

Classe A : H supérieur à 20 et H² x V½ supérieur à 1 500

Classe B : Ouvrage non classé en A et pour lequel H supérieur à 10 et H² x V½ supérieur à 200

Classe C : a) Ouvrage non classé en A ou B et pour lequel H est supérieur à 5 et H² x V½ supérieur à 20

b) Ouvrage pour lequel les conditions prévues au a ne sont pas satisfaites mais qui répond aux conditions cumulatives ci-après:

i) H > 2 ;

ii) V > 0,05 Million de m3;

Pour chacun de ces types de barrages ou de digues, une fréquence d'inspection et d'auscultation est donnée par le décret de 2015

	BARRAGE			DIGUE		
	Classe A	Classe B	Classe C	Classe A	Classe B	Classe C
Rapport de surveillance	Une fois par an	Une fois tous les 3 ans	Une fois tous les 5 ans	Une fois tous les 3 ans	Une fois tous les 5 ans	Une fois tous les 6 ans
Rapport d'auscultation	Une fois tous les 2 ans	Une fois tous les 5 ans	Une fois tous les 5 ans	Sans objet		

Comme on le voit, on distingue barrage et digues mais les biefs de navigation qui comprennent une section de canal comprise entre deux écluses sont assimilés par la réglementation de la sécurité hydraulique à des barrages, car ils peuvent libérer l'eau du bief lors d'une rupture accidentelle de leurs digues, dans les parties du canal en remblai.

Les différents types d'ouvrage auxquels VNF est assujetti à ce classement réglementaire sont :

- les barrages réservoirs qui assurent l'alimentation en eau des canaux à bief de partage : ces ouvrages au nombre de 50 ne posent pas de risque de sécurité dans la mesure où ils sont régulièrement contrôlés et où en cas de doute la cote de la retenue est abaissée tant que des travaux adéquats (comme le redimensionnement des évacuateurs de crue lorsque nécessaire est conduit. De plus une base de données assez complète dite de sécurité des ouvrages hydrauliques est menée.

	Classé par arrêté préfectoral	En attente d'arrêté préfectoral	Total
Classe A	6		6
Classe B	13		13
Classe C	13	2	15
Classe D	3	6	9
Non classé		7	7
Total	35	15	50

* Il faut observer que même si la classe D a été supprimée en 2017 pour ne garder que les trois classes du décret de 2015, le maintien de cette classification existe pour les ouvrages déjà classés

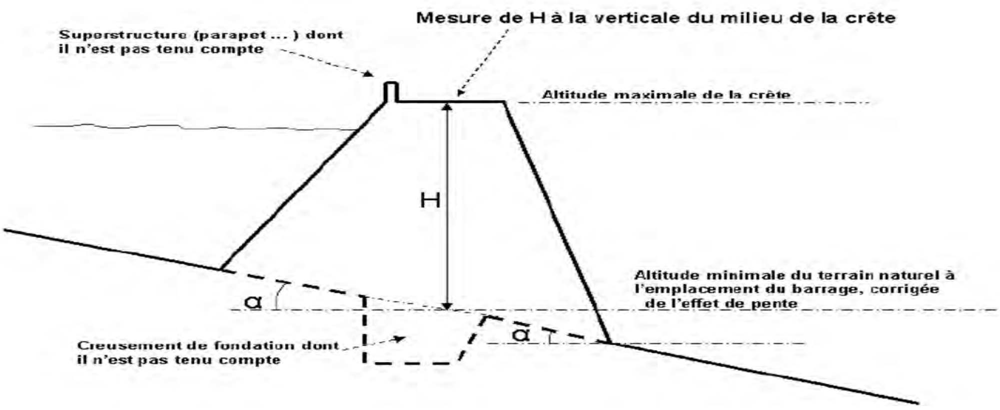
- les barrages de navigation (396) ont donné lieu à plus de difficultés puisque certains services de contrôle ont voulu prendre comme cote de la crête de ces barrages celle de la partie la plus élevée du génie civil de l'ouvrage, ce qui ne tient pas compte de la spécificité de la navigation. En voici un exemple pour le barrage de Suresnes sur la Seine: si l'on peut comprendre les précautions prises puisque ce barrage se situe au cœur de l'agglomération parisienne, on voit bien que ce choix de cote ne peut correspondre à une situation hydraulique réelle.

Un autre cas particulier de barrage de navigation est celui des barrages avec système de manœuvre ancien comme les barrages à fermettes et aiguilles dits créés en 1834 par l'ingénieur Poirée au moment du lancement du plan Becquey qui présentent aujourd'hui des risques sérieux pour les barragistes si bien que leur mécanisation ou leur modernisation devient indispensable mais dans un souci de sécurité de l'exploitant vis-à-vis de ses propres agents plus que vis-à-vis des riverains ou des usagers de la voie navigable.

- les digues des différents biefs (3756 km et 1799 écluses)

L'application de la réglementation est donnée par un arrêté d'application du décret de 2015 qui s'avère très contraignant pour VNF : en effet le classement est effectué pour la totalité du bief quelle que soit sa longueur et indépendamment du fait que certaines parties du canal sont en déblai.

Ainsi la façon de calculer la hauteur est donnée par le schéma suivant:



Ceci pose à l'évidence la question de devoir généraliser une approche sécuritaire indépendamment de la nature des risques réels encourus si bien que la mission a dû rappeler que l'approche devait être proportionnée aux risques réels afin de ne pas donner un systématisme dans l'auscultation et dans la surveillance aux zones qui ne le nécessitent pas.

- les tunnels-canaux : cas particulier mais important pour la sécurité des navires à passagers, les 31 tunnels canaux gérés par VNF développement un linéaire de 42 km (linéaire équivalent au linéaire des tunnels routiers du réseau national), et leur quasi-totalité (28) date des 18 ème et 19 ème siècles. La mission a estimé utile que VNF mette en place un dispositif national relatif à la maintenance qui s'appuie à la fois sur le référentiel CETMEF relatif à l'entretien, la surveillance et la réparation des tunnels canaux[10] et sur les recommandations techniques élaborées par VNF en 2013.

3- Financement de la politique de sécurité

La politique de sécurité de VNF est indissociable de sa politique de maintenance et de régénération du réseau, même si elle en représente une composante avec une logique propre par exemple lorsque l'autorité de contrôle de la sûreté hydraulique impose des travaux sur des ouvrages défectueux.

C'est ainsi qu'à la suite des crues du printemps 2016, VNF a présenté un programme très complet de 900M€ environ, réparti selon ses directions territoriales, qui comporte principalement une restauration de digues et une modernisation des barrages de navigation, des travaux complémentaires sur les barrages réservoirs et sur le système d'alimentation en eau des canaux et une composante importante de modernisation de la gestion hydraulique.

Nature d'opération par DT	**DTBS**	**DTCB**	**DTNE**	**DTNPC**	**DTRS**	**DTS**	**DTSO**	Total
Digues et berges	21,6	45,8	50,8	111,25	1,45	102	42,75	375,65
Barrages de navigation	181,18	43,0	66,0	4,2	11	11	0	316,38
Barrages réservoirs et alimentation	0	44,7	30	0	0	2	1,9	78,6
Instrumentation et ouvrages spéciaux	21,25	38,2	37,7	2,45	3,3	11,9	2,6	117,4
Total	**224,03 M€**	**171,7 M€**	**184,5 M€**	**117,9 M€**	**15,75 M€**	**126,9 M€**	**47,25 M€**	**888,03 M€**
Proportion	25,2 %	19,3 %	20,8 %	13,3 %	1,8 %	14,3 %	5,3 %	100 %

Comme ce programme qu'il serait bien de réaliser en dix ans, suppose un budget annuel qui est à peu près du double de celui dont dispose l'établissement, deux questions cruciales se posent pour lesquelles les bonnes pratiques de l'AIPCN apportent des enseignements précieux : la question du choix des priorités et celle de la mobilisation de ressources :

- pour les priorités, un travail préalable important avait été réalisé en 2009 pour établir les usages essentiels du réseau fluvial par itinéraire fluvial ce qui rejoignait les travaux menés au Congrès de

10 Tunnels-Canaux : fascicule 1 surveillance, entretien et réparation et fascicule 2 : pathologie CETMEF -2002 et VNF 2013 : Tunnels-canaux : fascicule de recommandations techniques pour la sécurité des tunnels-canaux

navigation de l'AIPCN à Osaka de 1990[11] et les recommandations du rapport InCom 139 plus récent [12] sur la valorisation des usages de la voie d'eau tandis que le second travail mené en 2016 par VNF s'est appuyé sur une méthodologie de priorisation par analyse de risques de tous les ouvrages concernés par la modernisation , la mise en sécurité et la conformité environnementale des ouvrages hydrauliques de son patrimoine. Ces choix sont tout-à-fait identiques à ceux préconisés dans les deux publications de référence de l'AIPCN sur le sujet[13] sur la gestion patrimoniale des voies navigables.

- pour la mobilisation de ressources financières, les formules adoptées par l'un des principaux gestionnaires de voies navigables historiques en Grande-Bretagne méritent d'être expertisées de même que celles qui s'efforcent de montrer que la navigation ne peut à elle seule supporter la gestion hydraulique que réalise aujourd'hui VNF sur les voies qui lui sont confiées, car elles contribuent au maintien du niveau des nappes hydrauliques, sont favorables au maintien de la bio-diversité des milieux aquatiques et participent de la fourniture de services écosystémiques.

4- Recommandations

Parmi les recommandations faites par la mission, nous en retiendrons quatre :

- la première concerne VNF pour l'utilisation de la base de données des ouvrages hydrauliques qui partant des barrages réservoirs doit assez rapidement pouvoir être étendue aux autres ouvrages, notamment les digues des canaux de navigation

- la seconde qui touche toujours VNF vise à mieux séparer dans son programme d'investissement la sécurité des ouvrages hydrauliques des autres types de sécurité(celle de ses propres personnels comme celle des usagers de la voie d'eau) car elle est directement liée au niveau d'appréciation des risques que l'Etat impose à tous les gestionnaires d'ouvrages hydrauliques et comme elle touche directement les riverains et les collectivités locales si bien que l'image de VNF peut en être directement impactée

- la troisième s'adresse à l'administration en charge de la prévention des risques pour qu'elle adresse à ses services de contrôle une instruction pour une application plus adaptée à la réalité des risques encourus sur les voies navigables, notamment pour la fréquence et le contenu des études d'auscultation et de surveillance des biefs de navigation qui découlent des classements des ouvrages opérés par la réglementation

- la quatrième enfin de portée plus générale pour amorcer une vraie réflexion sur l'avenir de la partie du réseau de VNF qui n'a plus aujourd'hui qu'une vocation principale de gestion hydraulique ou patrimoniale. En effet, VNF, après s'être assuré de la consistance et de la redondance du réseau principal à vocation commerciale (fret et touristique), pourrait engager avec toutes les parties prenantes une étude de reconfiguration des plus de 3000 km de voies fluviales concernées en examinant les différentes options possibles. La mission recommande aussi que les régions et les ensembles de communes en charge de la gestion des milieux aquatiques et de la prévention des inondations soient associées dès le début de ce travail et que les bonnes pratiques issues de problématiques internationales similaires soient mises à profit.

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11 Jean Maynadié, inspecteur général des voies navigables (AIPCN France) et Hans Peter Tzschucke, directeur construction du BAW (Bundesanstalt für Wasserbau)-(AIPCN Allemagne)-27ème congrès international de navigation-Osaka-Japon

12 PIANC report InCom 139-2016 Values of Inland Waterways

13 PIANC report InCom 129-2013 Waterway infrastructure asset maintenance management
PIANC report InCom 25-2006 Maintenance and renovation of navigation infrastructure